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FEASIBILITY STUDY AND COSTING OF PROPOSED POLLUTION CONTROL MEASURES IN THE HUMBER SEWERSHED

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Environment Ontario

Jim Bradley
Minister

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FEASIBILITY STUDY AND COSTING OF PROPOSED POLLUTION CONTROL MEASURES IN THE HUMBER SEWERSHED

Task 5 - Dunkers Flow Balancing System

Prepared for:
Toronto Area Watershed Management
Strategy Study (TAWMS)
Technical Committee

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TASK 5

FEASIBILITY STUDY AND COSTING OF PROPOSED POLLUTION CONTROL MEASURES IN THE HUMBER SEWER SHED

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TASK 5

1 - INTRODUCTION

An earlier study undertaken by our firm and forming part of the Toronto Area Watershed Management Strategy Study (TAWMS) covered four specific tasks related to the feasibility and costing of various pollution control works for the Humber Sewershed. In an effort to evaluate other potential solutions, this study has been undertaken, and is identified as Task 5.

The initial objective of this study was to evaluate the Dunkers Flow Balancing System (DFBS) as a contending substitute for the conventional detention facilities studied under Task 1 of the earlier study.

The DFBS was developed and patented in 1978 by Karl Dunkers, Stockholm, Sweden, and is based on the principle of detaining polluted water during times when flow exceeds sewer or treatment plant capacity. By utilizing pontoons and heavy duty reinforced plastic curtains, a detention basin can be created in a body of water such as a lake or wide river, as illustrated in Figure 1.

A series of bays can be created to control the flow of polluted water. The flow enters the first bay and is forced through each bay via an opening in the curtain. The flow displaces the clean lake or river water which was in the facility prior to the storm event. As the inflow subsides the stored polluted water is pumped to a trunk sewer or treatment plant. The stored water is therefore gradually replaced by the clean lake or river water and the system is once again ready for the next event.

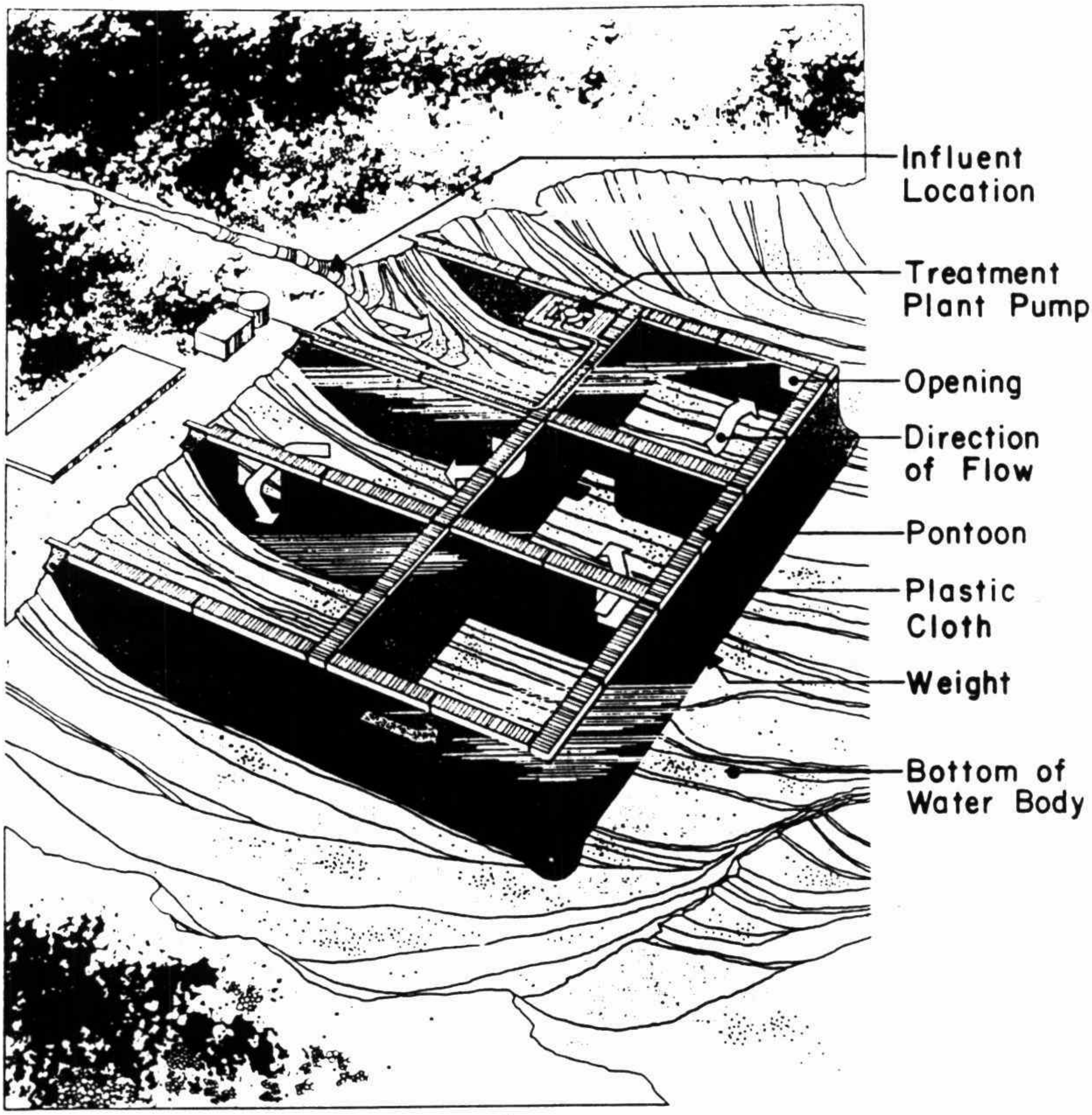
The advantage of the basic DFBS is that construction costs are less when compared to a detention facility on land,and there are no land requirements. The concept does, however, require a suitable in-water location.

2 - INITIAL INVESTIGATIONS

The initial objective of this study was to investigate the feasibility and costing to construct a DFBS as an alternative to the open basins or closed tanks which were studied in Task 1 of the overall study (Reference 1). The open basins or closed tanks are to be located adjacent to the existing regulators in the City of York sewer system and at Berry Road upstream of the Humber Water Pollution Control Plant (WPCP). The purpose of these facilities is to store Combined Sewer Overflows (CSO's) for return to the sanitary trunk sewer after the peak flows have subsided. The DFBS, if applicable, would then be installed either in the existing bodies of water (the Black Creek and Humber River) or in land at the same proposed locations as the open basins or closed tanks.

Field investigations and some initial calculations concluded that, due to insufficient depths and space limitations within these existing water bodies, the DFBS could not be feasibly located within either the Black Creek or Humber River. Furthermore, when preliminary estimates of the DFBS were prepared, it was found that the costs equalled or exceeded those established for the closed tanks of equivalent storage volume if the DFBS was to be constructed in land.

For these reasons, and following discussions with M.O.E. staff, an alternative use for the DFBS was found worthwhile an investigation, not as a flow balancing system, but rather as a flow through facility treating the heavily polluted flow from the Black Creek during low flow conditions. This type of system would make use of continuous flow to provide settling of materials analogous to a primary settling facility, with the effluent returned to Black Creek or Humber River, depending on the location. Disinfection could also be provided within the facility in order to reduce the bacteria levels.



Reprinted from Munters Corp., 1983[7]

Dunkers' Flow Balancing System

Artist's Impression

The Black Creek was selected, as a previous study (Reference 2) indicated that the level of many pollutants within the Black Creek exceeded the quality objectives under dry weather and wet weather conditions as set by M.O.E. By limiting the inflow rate from Black Creek, the facility can be designed to provide treatment of dry weater flows and the initial runoff during rainfall events which carry the most signiifcant pollutants from non-point soures. Although not a steady flow, there will be a small continuous flow through the facility during dry weather conditions, with a controlled maximum flow during wet weather, limited to retain settling velocities. The proposed facility would be operated continuously from May to October each year to coincide with the recreational season.

3 - OBJECTIVES

The objectives of this feasibility study have, for the reasons stated above, been revised to:

i) select a possible location for a primary treatment facility, utilizing the DFBS system as applicable;

ii) estimate the required size of the facility;

iii) outline the primary components of the facility, a possible operations plan and identify the more significant pollutants which could be treated;

iv) provide estimated capital and annual operation and maintenance costs for the facility.

As the DFBS concept is relatively new, information on technology, anticipated performance and lifespan is limited. The main source of material available is through the Canadian agent for the DFBS or from Mr. K. Dunkers. Appropriate references have been given when their data has been used in this report.

4 - LOCATION OF THE PROPOSED FACILITY

The proposed location of the DFBS is adjacent to the Humber River approximately 450 m south of the Black Creek (Figure 2). The lands on which the facility could be located are owned in part by the Metropolitan Toronto Regional Conservation Authority and by the Lambton Golf and Country Club. It is outside the scope of this study to investigate potential problems and costs associated with land acquisition.

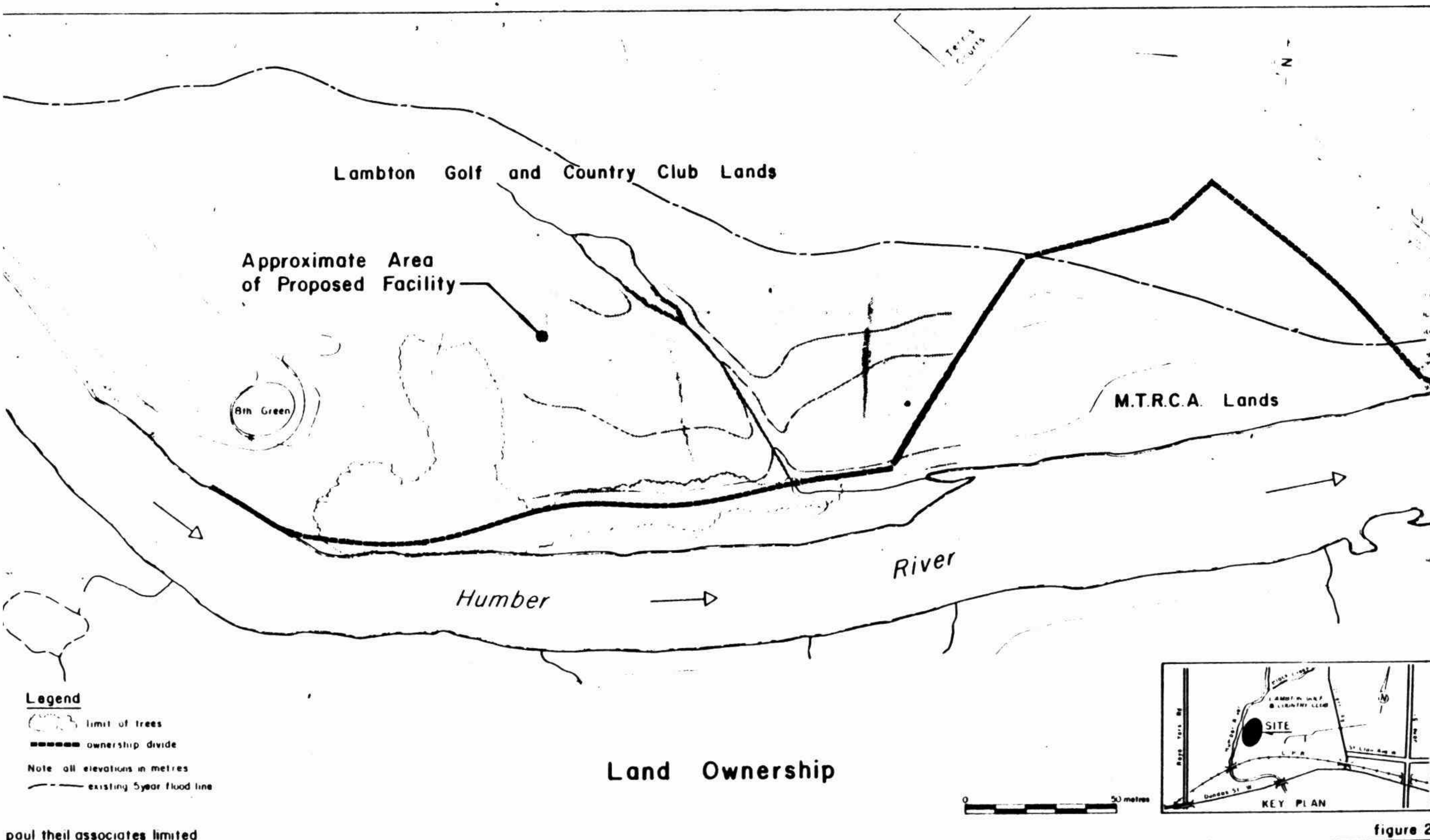
The reasons why this site was selected are as follows:

i) the land is located within the floodplain and is currently not in active use;
ii) the site has sufficient space to construct the facility;
iii) access to the site for construction and maintenance is available;
iv) sufficient drop in the water level between the inlet point and outlet point to allow the system to operate under gravity conditions;
v) the existing trees and escarpment surrounding the proposed facility will provide a visual screen betwen the golf course or the adjacent bicycle path on the west side of the Humber River;
vi) odour problems, if any, should be in the same order or less than what presently is experienced from Black Creek.

5 - PROPOSED FACILITY

5.1 Size

The DFBS has in this case been considered to be used as a dual facility, namely to: i) promote the settling of solids; and ii) provide disinfection of bacteria. The mechanisms of solids removal in the DFBS is similar to a conventional settling tank. The efficiency rate of settling the solids should, however, be superior as the flow arrangement of the Dunkers system will reduce the short circuiting effects which limit conventional settling



Land Ownership

figure 2

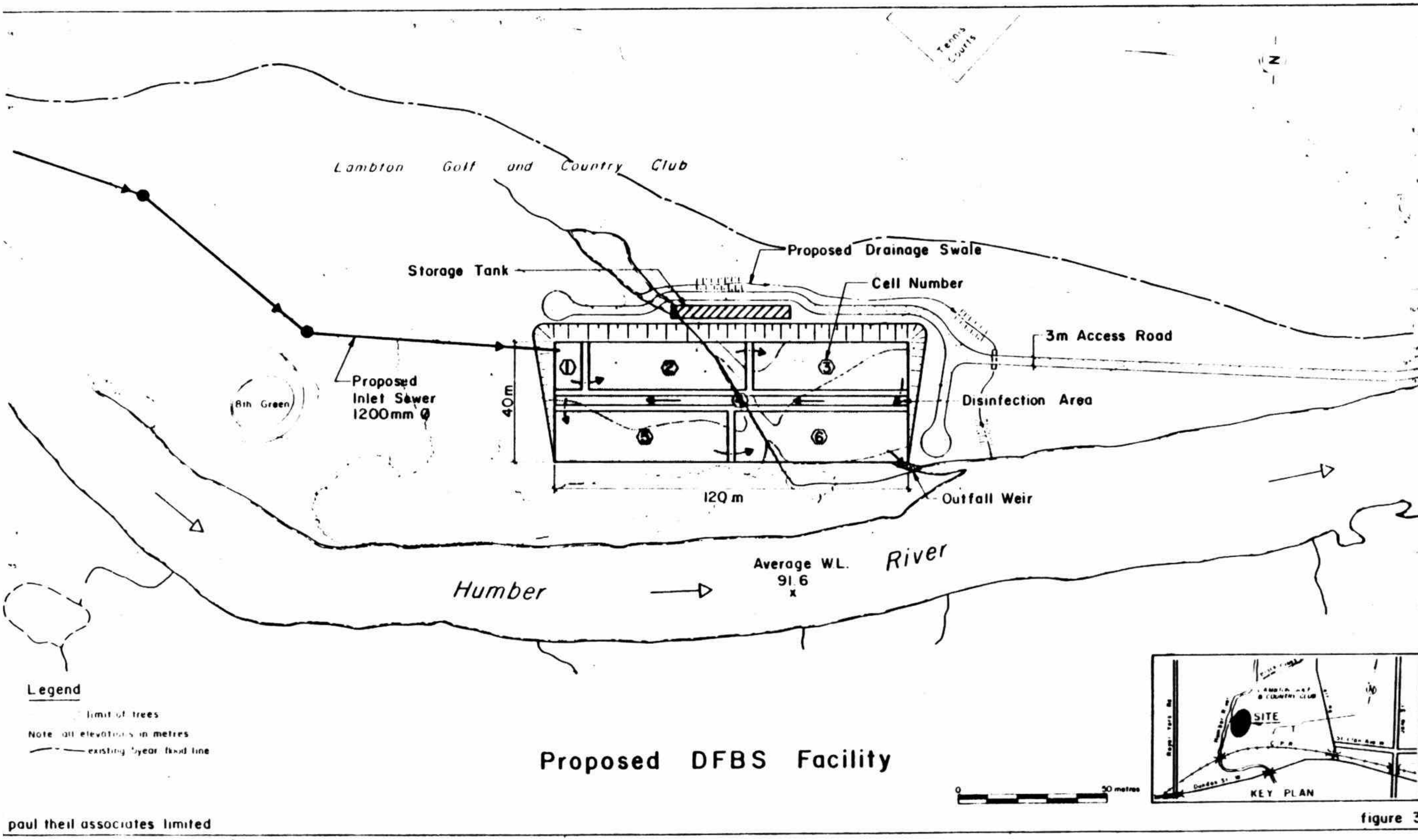
tank efficiency. In general terms, the facility will provide settling for the residue particulate (suspended solids) including metals which may settle out. Disinfection can be provided to reduce the concentrations of fecal coliforms and fecal streptococci.

The levels of various pollutants at the mouth of the Black Creek and in the Black Creek at Lawrence Avenue (a location where no CSO's exist) have been listed in Table 1. These levels are arithmetic means of several samples taken during the fall of 1982. Further information may be obtained from Reference 2. Both dry weather and wet weather levels are listed, as are the M.O.E. quality objectives for each parameter.

The facility could be operated at all times, except during freeze-up. For the purpose of estimating the volume of settleable solids, we have assumed the facility will be operated only during the period May to October, the same period applied for Tasks 1 through 4, as the objective would be to improve the water quality during the recreational season. Results from facilities already in place (Reference 3) indicate that although non-operative, the system may be left in place over the winter months without damage to its components.

The proposed size of the DFBS is based on two parameters: i) the flow rates within the Black Creek; and ii) the available land area within the propsoed site. In addition to the actual facility, space will be required for several appurtenances such as a holding tank for the sludge, storage room, access road for maintenance, inlet/outlet pipes to convey the flows to and from the proposed facility and a control house for any disinfectants. The proposed location and sizing for the DFBS and associated appurtenances is given in Figures 3 and 4. The southeast corner of the facility would coincide with the limit of the permanent easement for the Humber sanitary trunk sewer.

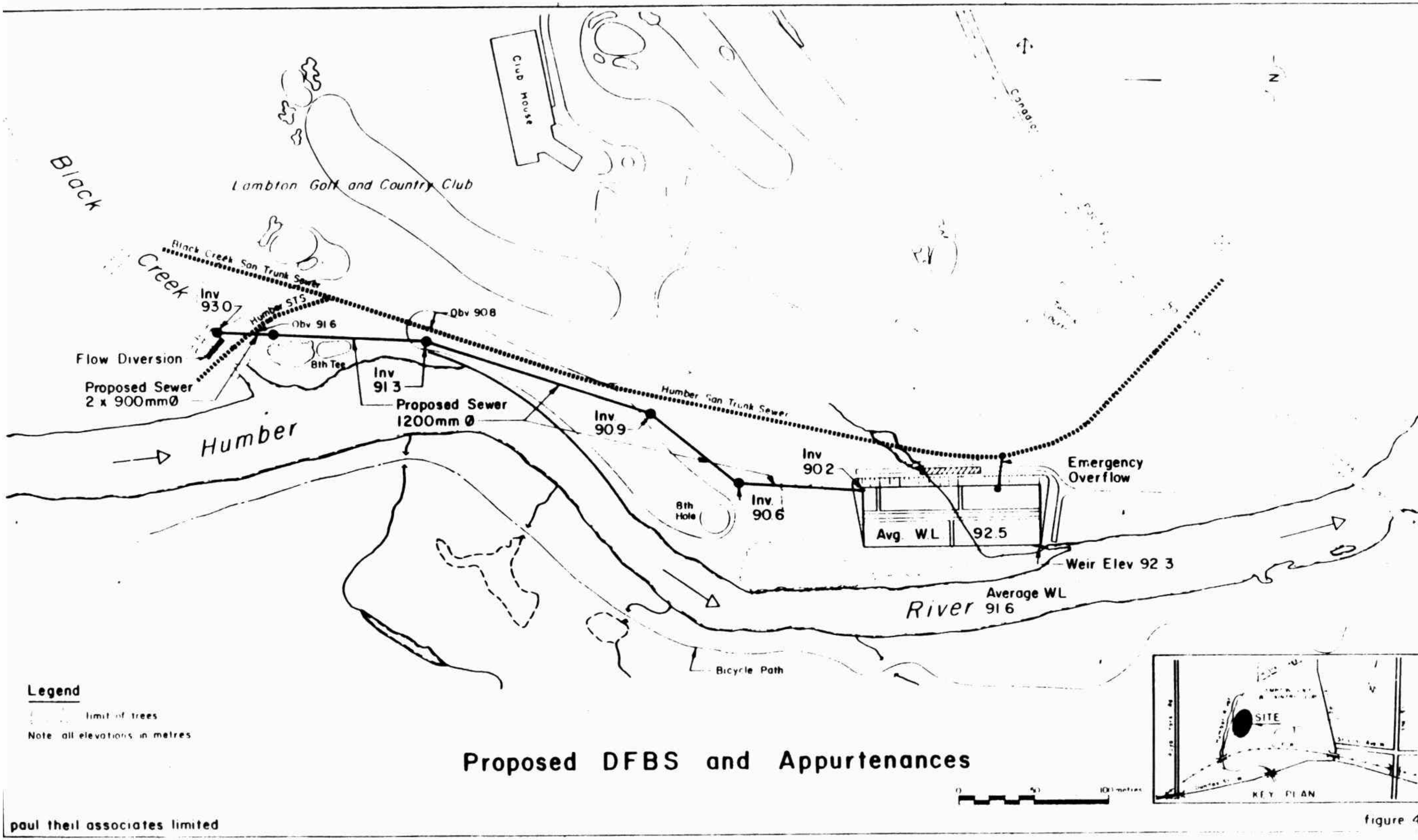
Streamflow records for the Black Creek at Scarlett Road were used as a basis to size the facility. The 1979 records were selected as this year was an average year in terms of precipitation and furthermore has been used by M.O.E. in other studies. The mean average flow for the months of May to October inclusive is 0.566 cms. (Source: Water Survey of Canada).



Proposed DFBS Facility

paul theil associates limited

figure 3



Proposed DFBS and Appurtenances

TABLE 1

POLLUTANT LEVELS WITHIN BLACK CREEK

Parameter (Units)	Station 11 - [a] Black Creek at Lawrence Avenue		Station 5 - [a] Black Creek at Humber River		M.O.E. Quality Objective
	Dry Weather	Wet Weather	Dry Weather	Wet Weather	
Residue Particulate (mg/l)	12.81	135.53	9.58	104.02	25.00
Copper (mg/l) [b]	0.014	0.023	0.018	0.029	0.005
Fecal Coliform (counts/100 ml)	783	1554	2418	9160 [c]	100
Fecal Streptococci (counts/100 ml)	247	3701	230	8903 [c]	-
BOD_5 (mg/l)	1.43	5.58	1.75	8.35	10.00

NOTE: a. Station numbers correspond to numbers given in Reference 2.

b. Other metals are also present. Copper was selected merely to illustrate the concentration of a heavy metal pollutant.

c. One or more values reported by the laboratory as "actual result is less than the reported value" were used to calculate this number. Consequently, this mean is higher than the actual mean.

As was stated previously, the function of the DFBS being considered in this study is similar, in principle, to a conventional settling tank. In order to size the facility the principles used in designing primary settling tanks were therefore applied. An overflow rate of 1000 Usgd/ft^2 (0.00054 cms/m^2) and a depth of 3 m was selected (Reference 4). Based on these parameters and the space limitations for the site, it was found that the proposed facility as shown in Figure 3 could provide satisfactory treatment for a flow rate up to 2.0 cms, or approximately four times the average daily base-flow.

This capacity will not be sufficient to treat all the flows for the May to October period. This value does, however, exceed 94 percent of the average daily flows for this period and therefore treatment of a significant percentage of the Black Creek flows will be attained.

5.2 Layout

The basic configuration of the facility is shown in Figure 3. The first cell will be used to remove the floatables and as an area for settling any large particles. Resuspension during the higher flows will likely occur in this cell. Cells 2 and 3 will provide settling of a majority of the settleable solids prior to disinfection. Disinfection will be provided in cell number 4 followed by two additional cells (5 and 6) for settling.

5.3 Inlet and Outlet Works

A preliminary route selection for the inflow and outflow pipes is shown in Figure 4, recognizing existing constraints such as trees, sewers and topography. The system will operate under gravity flow conditions. A small flow diversion weir (0.9 m in height) at the mouth of Black Creek will restrict flows from entering the Humber River. Twin 900 mm pipes will convey the flows above the Humber sanitary trunk sewer, then southerly to the proposed facility via a single 1200 mm pipe, a total distance approximately 450 m. Sufficient vertical clearance is available at the cross over point to prevent interference with the existing trunk sewer. The route as shown would across the eighth fairway of the Lambton Golf Course. This part of the works should preferably be constructed in the late fall.

The proposed invert of the 1200 mm pipe is above the obvert of the existing Humber trunk sanitary sewer. A minimum horizontal clearance of 3.0 m is available between the existing and proposed sewers along the eighth fairway.

The normal water level in the DFBS is proposed to be set at 92.5 m as controlled by the outflow weir. The five year flood level in the Humber River (93.8 m) exceeds the proposed outflow weir elevation, but is approximately the same as the land surrounding the DFBS. For storm flows exceeding the five year frequency, the facility should become ineffective, but due to the floats it should not suffer damage.

An emergency overflow to the Humber sanitary trunk sewer is recommended as shown in Figure 4. In this way, any chemical spills within the Black Creek watershed might be trapped and conveyed to the WPCP if desired. When the overflow is operating, the outlet opening from cell 3 would have to be closed.

5.4 Disinfection

By disinfecting the flow prior to outletting into Humber River, the fecal coliform and fecal streptococci levels from runoff within the Black Creek watershed can be reduced. The M.O.E. quality objective for fecal coliforms is 100 counts/100 ml. The data, as given in Table 1, shows that this level is significantly exceeded for both dry weather and wet weather flows, and not only from areas with combined sewers, but also from areas served by separate sewers. It is apparent that treating CSO only will not provide the necessary result; the measured pollutant levels at various locations of Black Creek point out that to obtain significant reduction, treatment of surface flows in Black Creek must be provided. Since the dry weather flows and the initial runoff from storms carry the heaviest concentrations, a basic treatment of these flows will no doubt be a cost-effective measure.

There are several methods available for disinfection. These include the use of chemicals such as chlorine, sodium hypochlorite and bromide chloride, or ultra violet (UV) treatment. UV treatment has been shown to be relatively cost effective (Reference 5) and the results of UV testing would seem

to indicate that a level of reduction sufficient to meet the water quality objective for fecal coliform and fecal streptococci could be attained. The most appropriate method can best be established at detailed design stage, but for the purpose of estimating costs we have selected UV treatment. The treatment efficiency and cost of the UV equipment is based on a maximum design flow of 2 m^3/sec. The costs were obtained from Trojan Industries Ltd., a well-known supplier of UV equipment.

5.5 Gravity Settling of Particles

The settling efficiency of the DFBS will be highly dependent on the settling characteristics of the suspended solids. For the purpose of estimating both the annual reduction in pollutant loading resulting from the proposed facility, and for estimating the quantity of sludge generated, a removal rate of 40 percent has been assumed. This rate is consistent with values used for primary settling tanks (Reference 4). This will not meet the M.O.E. quality objective of 25 mg/l of residue particulate during wet weather conditions. However, the pollutant loading to the Humber River will be reduced by approximately 95,000 kg per year. Furthermore, there will be an associated reduction in the pollutant loading of any heavy metals that settle with the residue particulate.

5.6 Sludge Removal

Based on the estimated volume of residue particulate (95,000 kg/yr), and using a value of four percent solids for the liquid sludge, the total volume of sludge generated per year will be about 1,000 m^3. Floating pumps have been used for removing the sludge in previous installations and are proposed for this facility in each of cells 1, 2 and 3. Using the above volume of liquid sludge, and assuming that a majority of the solids would settle in cells 2 and 3, it would be likely that the settled solids should be removed about four times per year. We propose a holding tank adjacent to the DFBS for temporary storage before disposal. If removal is carried out simultaneously with, or shortly after pumping from the cells, we do not expect any significant odour problems to occur, considering the rather isolated location.

6 - ESTIMATED COST OF PROPOSED FACILITY

Cost estimates for the remedial works as described in the previous section were prepared and are given below. The estimates were based on 1984 construction costs and include installation, construction, operation, maintenance, replacement and a 20 percent engineering and contingency allowance.

The estimates for the disinfection equipment were based on discussions with Trojan Technologies Inc. and should be taken as being preliminary estimates, subject to more detailed information being provided. Land costs (all works are located within the floodplain) have not been included. All annual costs were converted to capital costs using a 70 year time period and a net interest rate (interest rate minus inflation rate) of seven percent.

A detailed breakdown of the costs, quantities and a typical cross section through the facility is given in Appendix A.

The major capital costs include:

1. Excavation and disposal.
2. Hand laid rip rap.
3. Pontoons and curtains for the DFBS.
4. Inlet pipes and outlet works.
5. Storage tank facility for the sludge.
6. Sludge removal equipment.
7. Disinfectant equipment.
8. Landscaping.

The primary factors used in sizing the UV equipment were: the incoming bacteria and suspended solids levels, quality objective values for the bacteria, and suspended solids and the flow rates.

In addition, annual costs for operation, maintenance and replacement of the equipment which were considered. The expected lifespan of the pontoons and curtains is 35 years (Reference 3). A five year lifespan was assumed for

the sludge removal equipment. An annual cost was included for the replacement of the UV lamps (a lifespan of two years can be expected). For the sludge storage tank a 70 year life span was selected. Annual costs for removing the settled solids from the facility and trucking the liquid sludge to the Humber WPCP have been included.

The capital cost for constructing the facility, including inlet and outlet works, was estimated to be $1,798,000. The annual operating and maintenance costs were found to be $545,000, capitalized over 70 years. The total cost to construct and operate the facility over a 70 year life span would therefore be $2,343,000. Table 2 (Appendix A) gives a breakdown of the major cost components. The capital costs for the pontoons, curtains and sludge removal equipment were obtained from the Canadian supplier for the DFBS (Reference 6).

7 - CONCLUSIONS

1. The Dunkers' system, in its traditional sense as a flow balancing system within a body of water, was found not to be feasible in either the Humber River or Black Creek. Furthermore, to construct a DFBS on land to store CSO's is not cost effective. For these reasons, the Dunkers system was investigated as a possible flow through facility, treating polluted flows from the Black Creek.

2. The flow in Black Creek, primarily during dry weather and at the initial runoff stage of rainfall events, is heavily polluted both in areas with separate sewers and in areas served by combined sewers. It is apparent that treating CSO only will not provide any substantial improvements to the water quality in the Humber River. To achieve this, treatment of surface runoff will have to be provided, particularly in the Black Creek watershed. The primary treatment provided by a basin of the type similar to a DFBS will be cost-effective.

3. The proposed location of the DFBS is adjacent to the Humber River some 450 m south of Black Creek. The facility will be about 40 m x 120 m x 3 m deep, providing a capacity of 14,400 m^3. The average flow through the facility will be approximately 0.56 m^3/sec, the peak flow about 2.0 m^2/sec. The facility is of sufficient size to treat the average daily flows 94 percent of the time for the May to October period.

4. The facility, including disinfection equipment, will signficantly reduce the suspended solids loading to the Humber River from the Black Creek. Furthermore, the fecal coliform and fecal streptococci levels will likely be reduced below the M.O.E. quality objective values.

5. Unlike at present, the proposed facility will provide an opportunity to capture accidental surface spills of hazardous liquids within the Black Creek watershed, with a provision to convey the spill through the Humber WPCP, if and when desired.

6. The preliminary estimates indicate that the facility can be constructed for $1,798,000. Annual costs, capitalized over a 70 year period, will be an additional $545,000.

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4. "Process Design Manual for Suspended Solids Removal". United States Environmental Protection Agency, January 1985.

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6. Canadian Effluent Treatment Inc. "Cost Evaluation of a Horizontal Flow Balancing System For Harbour Slips". A report to the Ministry of the Environment, February 8, 1985

7. Munters Corporation. "Dunkers' Flow Balancing System for Combined Sewer Overflow, Urban Runoff, Industrial Discharges". Brochure dated February 1983.

TABLE 2

APPENDIX A

BREAKDOWN OF ESTIMATED COSTS AND QUANTITIES

Inlet Works

1 - 1200 mm sewer	405 m @	387.00	=	$ 156,730.00
2 - 900 mm sewer	40 m @	498.00/m	=	19,920.00
6 - 1500 mm manholes	6 @	3000.00 each	=	18,000.00
Inlet/diversion structure		Lump sum	=	23,000.00
Restoration	12,000 m^2@	3.00/m^2	=	36,000.00
				$ 253,650.00
Engineering and contingencies 20%				50,730.00
				$ 304,380.00

TABLE 2 - Cont.

APPENDIX A

BREAKDOWN OF ESTIMATED COSTS AND QUANTITIES

DFBS Facility and Appurtenances - Capital Costs

Excavation & disposal	19,200 m^3	@	5.50/m^3	=	$ 105,600.00
Rip rap	1,810 m^2	@	25.00/m^2	=	45,250.00
Pontoons	308 m	@	675.00/m	=	207,900.00
Curtains	308 m	@	750.00/m	=	231,000.00
UV equipment				=	500,000.00
Sludge storage tank/ removal equipment	250 m^3			=	105,000.00
Miscellaneous, access road, hydro, outlet works, landscaping				=	50,000.00
					$ 1,244,750.00
Engineering and contingencies 20%					248,950.00
					$ 1,493,700.00

DFBS Facility and Appurtenances - Annual Costs (capitalized)

UV lamp replacement (2 year life span assumed)	=	$ 250,000.00
Pontoon/Curtain/Sludge Pump replacement	=	60,000.00
Removal and transportation of sludge to WPCP	=	112,000.00
Power	=	123,000.00
		545,000.00

TOTAL ESTIMATED COST	$ 2,343,080.00

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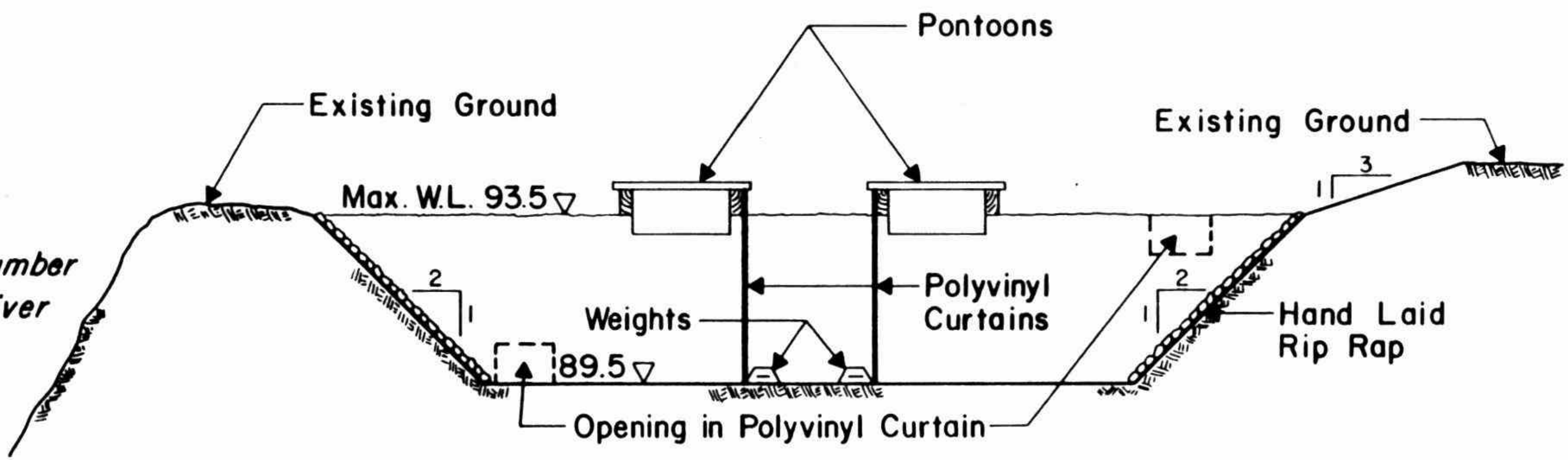

East - West Cross - Section through Proposed Facility

not to scale

figure A-1

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FEASIBILITY STUDY AND COSTING OF PROPOSED POLLUTION CONTROL MEASURES IN THE HUMBER SEWERSHED

Task 6 - Dry Weather Flow Interception and Treatment

Prepared for:
Toronto Area Watershed Management
Strategy Study (TAWMS)
Technical Committee

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December 1988

FEASIBILITY STUDY AND COSTING OF
PROPOSED POLLUTION CONTROL MEASURES
IN THE HUMBER SEWERSHED

TASK 6

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TASK 6

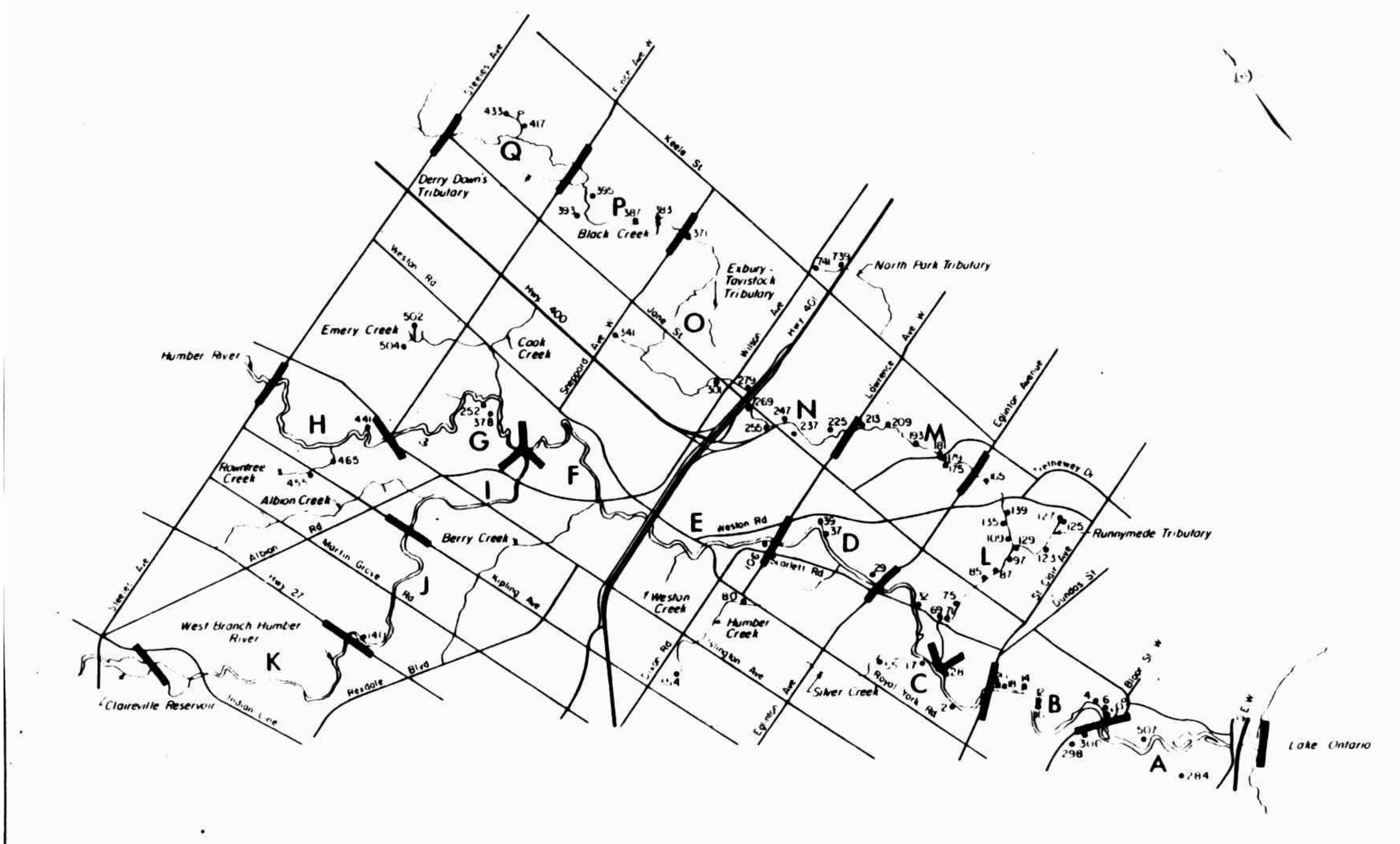
1 - INTRODUCTION

A recent study was undertaken for the Ministry of the Environment to investigate dry weather flows (dwf) from sewer outfalls to the Humber River and its tributaries (Ref. 1). As a result, discharges from 68 outfalls have been identified as warranting abatement action due to their relative bacteriological and chemical contamination. Interception of these discharges for treatment will reduce pollutant loads to the receiving waters. Therefore, the purpose of this study is to determine the technical feasibility of dwf interception and to provide estimated costs for interception works and treatment. This report has been prepared as Task No.6 of the study titled "Feasibility and Costing of Proposed Pollution Control Measures in the Humber Sewershed", prepared by Paul Theil Associates Limited for the Toronto Area Watershed Management Strategy Study (TAWMS).

The outfalls of concern are distributed along the Humber River and its tributaries. A majority of the outfalls are located along the lower reaches of Black Creek and Humber River. The location of each outfall investigated in this study is shown in Figure 1.

The dry weather flows to be intercepted primarily discharge from storm sewers (some outfalls were found to be combined sewer outfalls) and as such the following constraints were considered:

1. Intercepted dwf rates should not exceed flow rates observed during recent field tests and should not increase during wet weather.



Feasibility Study And Costing Of

Proposed Pollution Control Measures In The Humber Sewershed

OUTFALLS INVESTIGATED FOR DRY WEATHER FLOW INTERCEPTION

Original by Gartner Lee Associates Limited

legend

▬	reach boundary
A	reach name
100	outfall number

FIGURE 1

2. The hydraulic properties of the existing sewer upstream of the point of interception should not change.

3. Interception of dwf should not violate existing sanitary sewer use by-laws.

4. Conveyance of dwf to the WPCP should be by cost-effective means.

2 - METHODOLOGY

The steps used to meet the objectives of this study are listed below:

1. All maps and plan and profiles available for each outfall and of the Metropolitan Toronto sanitary trunk sewer were collected and reviewed.

2. Various alternative methods of dwf interception were investigated.

3. Technical feasibility was assessed for each location by selecting the most appropriate method of interception. The requirements for each method of dwf interception selected are presented in Section 4.

4. Intercepted dwf for feasible sites were reviewed to ensure compliance with the existing Metropolitan Toronto sanitary sewer use by-laws.

5. Cost estimates for interception works at each technically feasible site were prepared.

6. Total flows intercepted during dry weather were estimated to obtain the associated treatment costs.

Costs for engineering works and treatment are summarized by reach.

For each case the most cost-effective location for interception was identified. In some cases the location was identified some distance upstream of the outfall. For each case the percent of dwf capture was estimated by approximating the catchment area.

For the purpose of this study the existing trunk and sanitary sewers which are to convey the intercepted flows were assumed to have sufficient capacity. This can only be verified, however, by extensive monitoring and detailed hydraulic analysis.

Illustrations were prepared for each method of capturing dry weather flows and for each feasible site to show the location of proposed works.

A table showing the estimated loading of each parameter for each reach investigated in this study is provided. This table indicates the potential load reductions feasible through dwf interception.

3 - DATA COLLECTION

In order to assess the technical feasibility of dwf interception, information and data were obtained from sewer maps and plan and profile drawings provided by the Works Departments of the municipalities of Etobicoke, North York, Toronto, York and Metropolitan Toronto. Copies of the available drawings were obtained far enough upstream of the outfall to assess if a more cost-effective location of interception was feasible other than at the outfall. These drawings provided information on location and elevation of the storm and sanitary sewers as well as information on land use, easements, property lines and adjacent utilities such as gas, hydro and water.

Plan and profiles were available for all but four outfalls. These outfalls (#55, 455, 80 and 209) were not located on any of the municpalities' sewer maps or drawings. Feasibility of interception was therefore not assessed for these outfalls. Unfortunately some of the drawings suffered from insufficient data, poor quality due to age of originals, and lack of updating to reflect later revisions. In this case, if possible relative inverts were extrapolated or interpolated from other drawings to assess feasibility. Several pipe sizes did not agree with those found in the outfall report. However, if sufficient data was available, feasibility was assessed on the data provided in the drawing.

Prior to any works, field investigations should be carried out to verify inverts, pipe sizes and lengths.

4 - METHODS OF INTERCEPTION

The flow rates observed during the dry weather survey in general are very small, ranging from 0.01 to 50 l/s with a total flow of 319.29 l/s and a mean maximum rate of 4.7 l/s. The largest observed flow rate from a series of measurements at a given outfall was used as the flow rate to be intercepted. The observed range of flows for each outfall are shown in Tables 3a - 3d (see Section 7).

Various methods of dwf interception were reviewed and evaluated with respect to hydraulic requirements, includng:

1. Ability to intercept low flows without significant increase during wet weather.

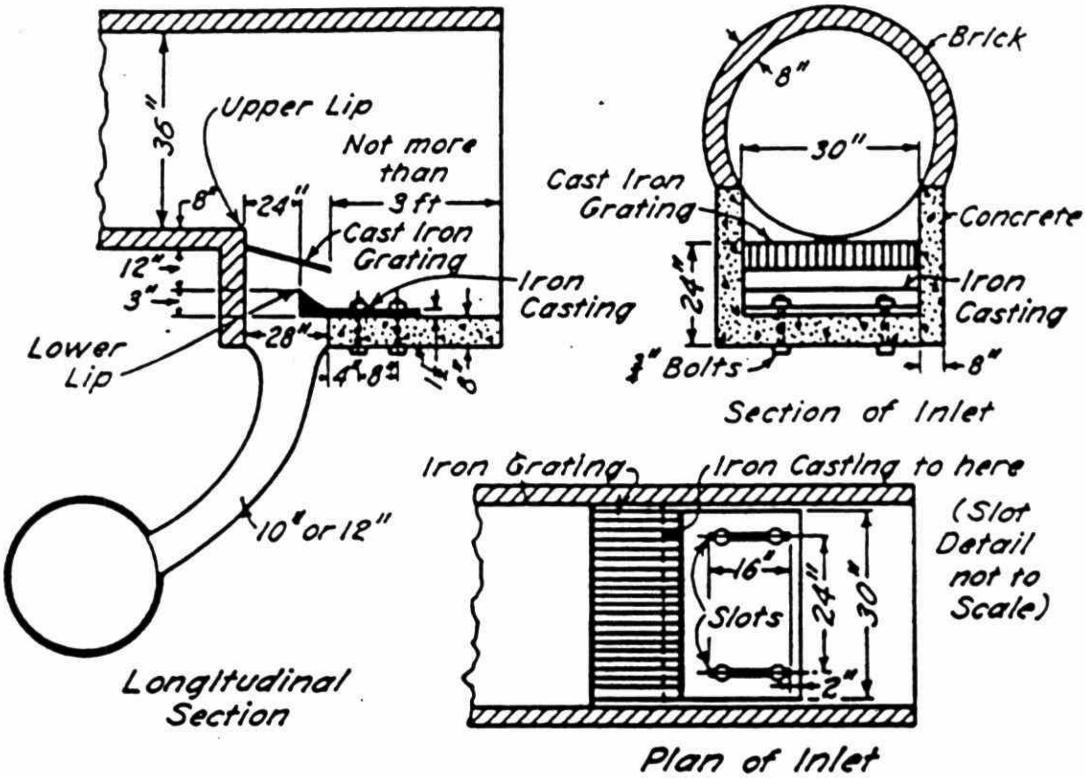
2. Simplicity and reliability of operation; static devices with no moving parts utilizing available grade between storm and sanitary sewer are preferred.

3. Simplicity of maintenance; devices prone to clogging should be avoided wherever possible.

As a result of this evaluation, three basic methods of dwf interception were selected for the range of conditions found. The first method takes advantage of available sewer hydraulics, the second employs a static hydraulic device and the third method is based on pumping. Each method is discussed below with a general description of their operation, as well as a summary of their basic advantages and disadvantages.

4.1 Method 1 - Leaping Weir

A leaping weir is formed by a gap in the invert of a sewer through which the dwf will fall. The greater velocity during higher wet weather flows causes the flow to leap over this gap. Figure 2 illustrates an example of a typical installation (Ref. 2).



EXAMPLE OF LEAPING WEIR

SOURCE: H.E. BABBITT - "SEWERAGE AND SEWAGE TREATMENT"

FIGURE 2

The use of leaping weirs are governed by the available grades of the sewer system. In order to create the necessary leap, a drop in pipe invert is required. If insufficient drop exists at an existing manhole, one may be created by using a larger outlet pipe at a flatter grade. A leaping weir installation in a manhole is illustrated schematically in Figure 3. A set of equations have been determined from tests on leaping weir design to describe the curved surfaces of the falling stream (Ref. 2). These equations are based on a minimum invert drop of 153 mm. This was used as a criteria for the use of leaping weirs in this study.

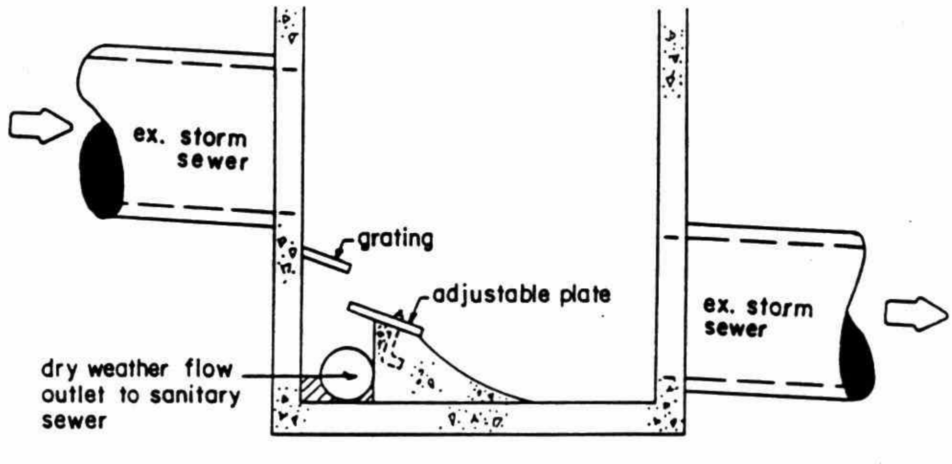
To allow for gravity flow, the sanitary sewer must be lower than the invert of the storm sewer. Accordingly, leaping weirs are recommended only for those sites where the sanitary sewer obvert was found to be at or below the storm sewer invert at the point of interception. This will provide reasonable protection from potential contamination from possible surcharge of the sanitary sewer. As a further protection, a backflow valve can be installed.

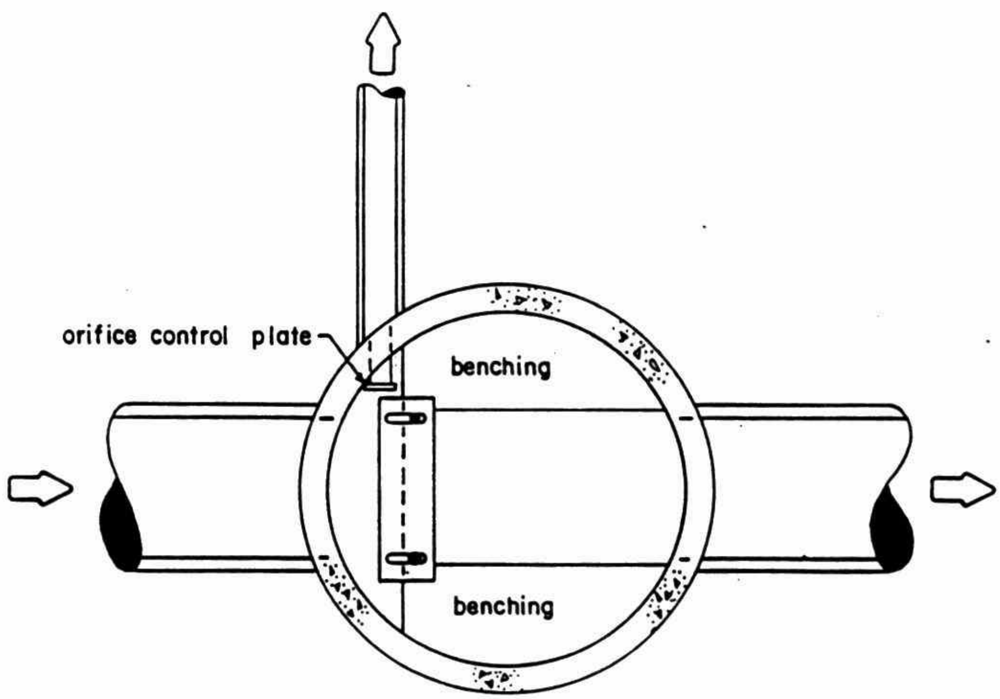
A leaping weir has the advantage of operating as a regulator with no moving parts, as well as the flexibility of controlling the quantity of flow intercepted by adjusting the gap opening. It is able to intercept the very low flows which are found in the sewer outfalls covered by this study and with minimal effect on upstream hydraulics. In some cases a small sump will be required to develop the head to convey the dwf as shown in Figure 3.

A disadvantage of a leaping weir is the potential for intercepting more than the intended flow if the storm sewer outfall should be subject to surcharge. In such cases, the intercepted flow will be governed by the surcharging head. However, the frequency of such occurrences is expected to be low and, by using a suitable orifice on the outlet to the interceptor, a flow limit can be created. The grates are susceptible to blockage by leaves, paper, etc. However, this can be minimized by adequate slope to the grate.

4.2 Method 2 - Hydro-Brake

A hydro-brake is a device which self-acts to limit liquid flow. The principle of operation is to attenuate and/or limit flow by converting flow into





LEAPING WEIR IN MANHOLE
(schematic)

FIGURE 3

a centrifugal motion which effectively reduces the forward velocity of the flow. The result is a greater reduction in flow for a given head compared to an orifice. A comparison of capacities of four types of hydro-brakes to that of an orifice is shown in Figure 4. The conical in-line design was selected for this application, since it requires the least amount of head to convey the dwf. Hydro-brakes can be used at locations where storm sewer grades are insufficient for the use of leaping weirs. A typical hydro-brake installation in a manhole is shown in Figure 5. Hydro-brakes have been applied in many stormwater management applications, and have a record of being relatively maintenance free.

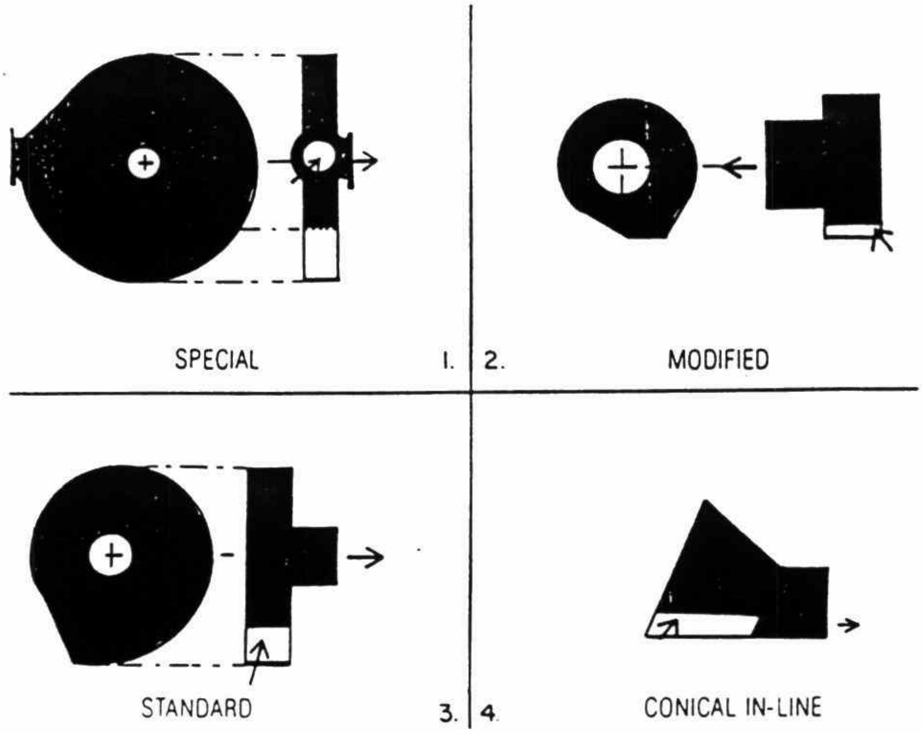
In order for the hydro-brake to operate during dry weather conditions, some manholes will require a sump to provide the necessary head to convey the flow. The addition of this sump will have minimal effect on the hydraulics of the manhole. To minimize the potential for the inlet becoming silted in, the hydro-brake should be installed at a height above the manhole invert to accommodate sediment build up between regularly scheduled maintenance periods.

The main advantage of hydro-brakes are that they can reduce the amount of wet weather inflow considerably without restricting dwf interception. They have no moving parts, consume no energy and require minimal maintenance. They come in a range of sizes and are custom made from stainless steel.

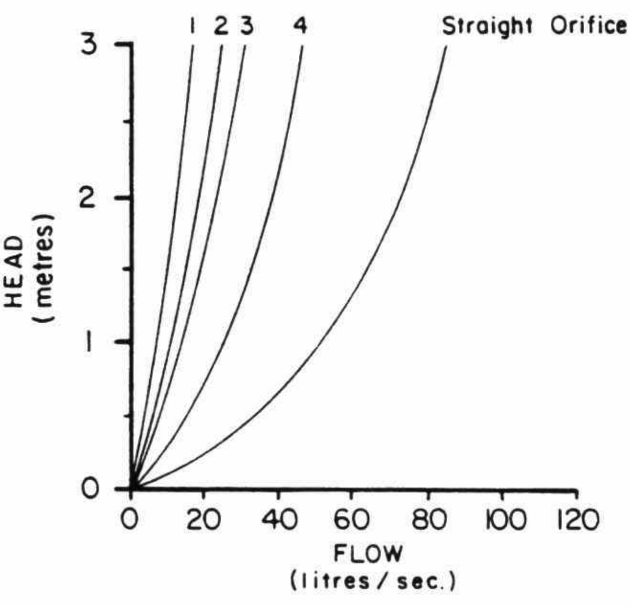
The disadvantage of using hydro-brakes for dwf interception is that even though they greatly limit the volume of wet weather inflow, they will permit some portion of wet weather flows to enter for each event, the extent of which can be determined only during detailed design.

4.3 Method 3 - Sump Pump

At sites where sanitary sewers are above storm sewers and downstream connections to the Metropolitan Toronto trunk sewer are not feasible, the use of sump pumps is suggested. By using a manhole as a wet well, a single submersible sump pump can readily be installed with float switches. The set-up would be rather similar to that used in homes with sump pumps in the



HYDRO-BRAKE STYLES

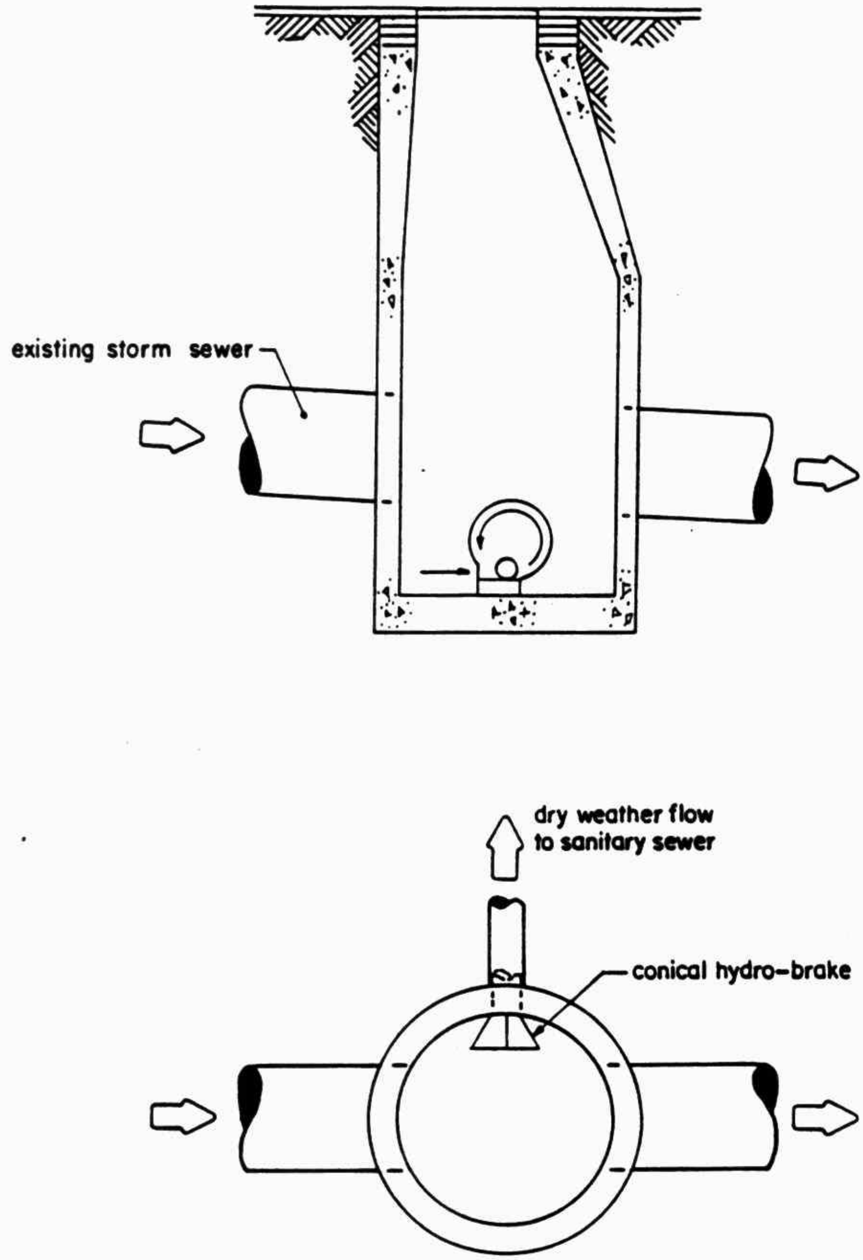


HYDRO-BRAKE PERFORMANCE CURVES

(FOR 150mm OPENING)

SOURCE: HYDRO STORM SEWAGE CORPORATION - "HYDRO—BRAKE LIQUID FLOW CONTROLS"

FIGURE 4



HYDROBRAKE REGULATOR IN MANHOLE

(SCHEMATIC)

FIGURE 5

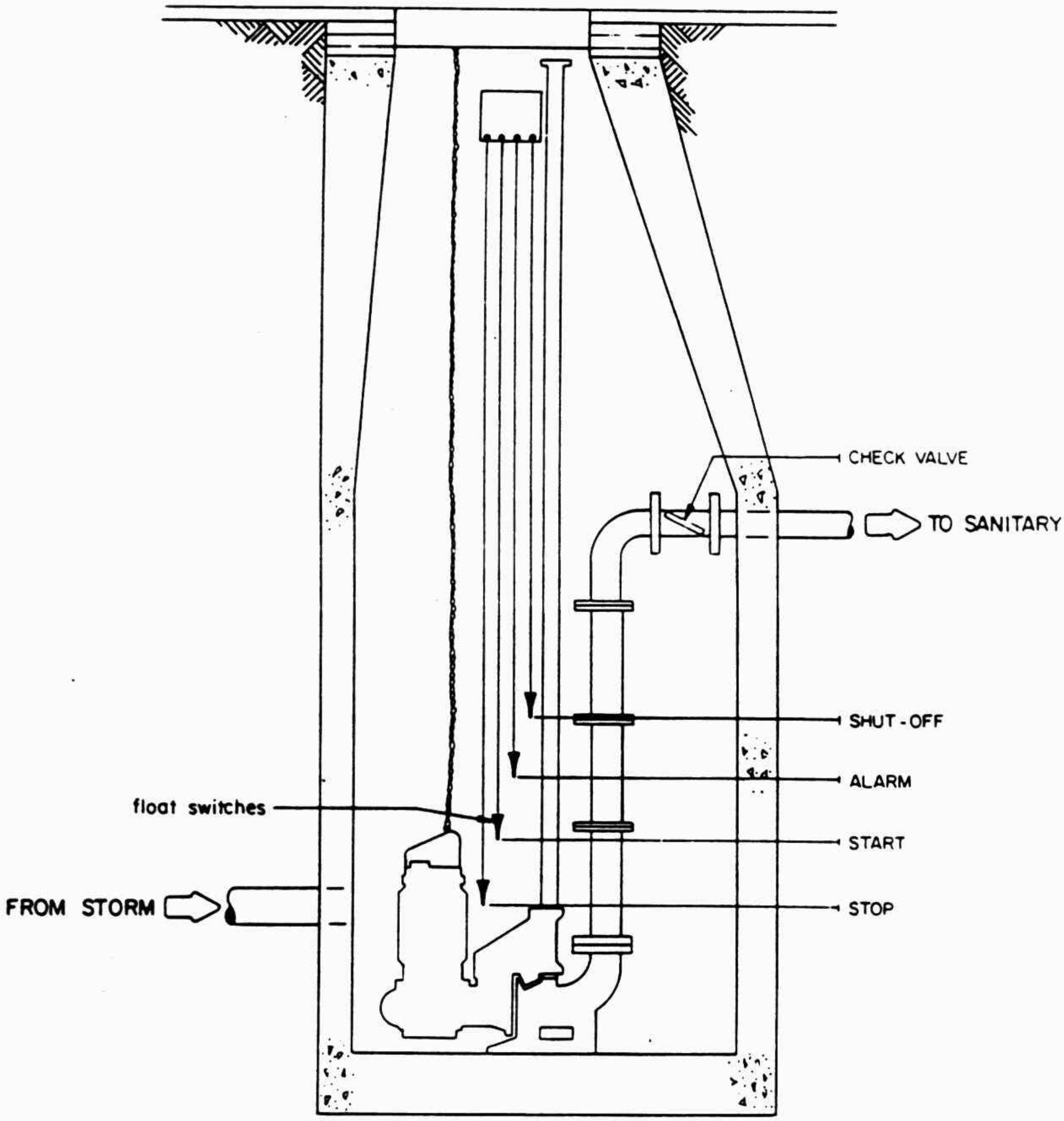
basements. The use of an additional pump for back-up should not be necessary, since an occasional pump failure will only cause a temporary delay in dwf interception. A typical type of intallation is shown schematically in Figure 6. The basic operation of this system is as follows.

Under dwf conditions the water level in the sump will fluctuate between the start and stop float switches, with the start float set at an elevation just below the storm sewer invert. During wet weather the automatic shut-off switch, which is set at an elevation just above the maximum dwf design level, will shut the system off. When the combined dry and wet weather flows are equal to the pumping rate, small volumes of wet weather flow will be intercepted. When the high flows subside the system returns to dwf operation. To inform operators of levels higher than dwf, either due to wet weather conditions or pump failure, an alarm switch can be installed, if desired. For protection against backflow from a surcharged sanitary sewer, a check valve is required. Remote controls can also be installed for additional flexibility.

Although a sump pump is a mechanical device, obviously requiring more maintenance than a static device, they do provide a reliable and flexible means of flow control. In any event, pumping is required where the storm sewer is below the sanitary sewer.

The disadvantages of a sump pump installation is the greater maintenance and capital cost compared to those for leaping weirs and hydro-brakes. However, such systems are relatively simple, reliable and basically trouble free.

For the purpose of inspection and maintenance, each intercepting device should be installed in a manhole. For some sites existing manholes may be used and in other cases new manholes will be required. Existing manholes may require modifications such as removal of benching and/or the installation of sumps. A manhole is generally not required where connection is made to the sanitary sewer.



SUMP PUMP IN MANHOLE

FIGURE 6

5 - ASSESSMENT OF FEASIBILITY

Feasibility for each of the outfalls being considered was assessed, based on the level of compliance to the constraints outlined in Section 1.

In order to select the most appropriate method for a given site, the following basic variables were investigated:

1. Sanitary sewer invert elevation relative to the storm sewer.

2. Sanitary sewer location relative to the storm sewer.

3. Existing storm sewer grade or the presence of a drop in pipe inverts at a manhole.

In most cases, interception can be provided upstream of the outfall where the sanitary sewers are generally close by, thus minimizing the length of intercepting sewer required, and where 100 percent capture is possible. Direct connections to the Metropolitan Toronto sanitary trunk sewer have been avoided where possible.

Where storm sewer inverts were above the adjacent sanitary sewer obvert, the use of a leaping weir was considered first. If found not feasible, then the hydro-brake method was selected. The size of hydro-brake has been based on the rate of flow to be intercepted. As a minimum diameter for openings for orifices plates or hydro-brakes, we have selected 90 mm, since through experience this has been found to be the smallest practical size that will provide a minimum potential for flow obstruction by debris. This diameter also meets dwf requirements.

At each site the potential peak inflow was also estimated. For leaping weirs and hydro-brakes this flow rate was calculated by assuming full pipe flow in the storm sewer. The largest pipe size was used at locations where pipes increase in diameter, or when more than one pipe entered a manhole. For leaping weirs, the peak flow was calculated assuming that a 90 mm

orifice was installed downstream of the leaping weir. For hydro-brakes, head to discharge curves were used. For sites using pumps wet weather flow rates will be a function of the set pumping rate and cycle time. For this study, peak flow rates were estimated using a 3:1 pumping to dwf rate.

In order to quickly review the relative cost-effectiveness of the various installations, a low, medium or high level of feasibility was identified for each outfall, based on their relative capital costs. Those outfalls that incur disproportionately high capital costs have been classified as low feasibility. Conversely, those with relatively low capital costs have been classified as high feasibility.

Level of feasibility was not assessed on the quantity of dwf intercepted, as the objective of interception is to reduce pollutant loadings to the receiving water which is a function of flows intercepted as well as concentration. Note level of feasibility does not include the cost associated with treatment which is also a function of the flow rate intercepted.

6 - BY-LAW CONSIDERATIONS

The feasibility of dwf interception was reviewed with respect to discharge and water quality compliance to municipal bylaws. The Metropolitan Toronto bylaw reads as follows:

"No person shall discharge or deposit or cause or permit the discharge or deposit into a sanitary sewer, combined sewer, public or private connection to any sanitary sewer or combined sewer, storm water or uncontaminated water, except which may be discharged into a combined sewer, unless the discharge into a sanitary sewer is permitted by the area municipality."

Of the Cities contacted, Etobicoke, North York and York reported that storm water is not permitted into sanitary sewers. The City of Toronto bylaw does permit storm drainage from a municipally owned storm sewer to discharge into a sanitary or combined sewer, but does not permit discharge from building storm sewers. For the other Cities, the basis of non-acceptability is primarily due to the conception that storm water is uncontaminated and, therefore, considered an extraneous flow not requiring treatment. A second concern is that of existing sewer capacity for the protection against basement flooding. Metropolitan Toronto's concerns are of treating additional flows and also existing sewer capacity.

Since intercepted dwf originating from the storm sewers contains contaminants, discharge to the sanitary sewer was checked to ensure compliance with the Metropolitan Toronto sewer use by-laws (Ref. 3), since treatment will be at Metropolitan Toronto's Humber River Water Pollution Control Plant.

Table 1 lists the by-law limits for those parameters which are of specific concern to the Humber River and for which data was available from the Ministry of the Environment dwf study. Note that no limit exists for the discharge of fecal coliforms to the sanitary sewer, therefore no bacteriological violations will occur.

TABLE 1

SANITARY SEWER BY-LAW LIMITS

Parameter	Metropolitan Toronto Sewer Use By-law (mg/l)
Chromium	5.0
Copper	5.0
Iron	50.0
Lead	5.0
Zinc	5.0
Mercury	0.1
Phenolic Compounds	1.0
Fecal Coliforms	None

Ref. 3: The Municipality of Metropolitan Toronto Bylaw No. 148-83.

Outfalls which were found to have dwf discharges in violation of the Metropolitan Toronto sewer use by-laws are listed in Table 2. Nineteen (19) of the feasible interception sites were found to have at least one sample in violation of the phenolic compounds limit, and some had as many as five violations. One violation of zinc and mercury was also found for outfalls numbers 502 and 140 respectively.

7 - FINDINGS AND RESULTS

Information from available maps and drawings were used to assess the feasibility of providing means to intercept dwf. In some cases, however, a plan and profile may not have the required sanitary sewer information, but the location of the nearest sanitary sewer was available on general sewer maps. This information was then transferred to the plan and profile. For each feasible site, a copy of the pertinent portion of the plan drawing is provided in Appendix A, Figures A1 to A43. Additional information has been added where necessary. The area of the proposed interception works is circled. A table on each drawing indicates the selected method of interception, existing storm and sanitary sewer inverts, sizes and grades, and the length and size of the proposed intercepting pipe.

Information for each outfall is presented in summary form in Tables 3a to 3d stating:

1. Municipality
2. Reach
3. Outfall number
4. Drawing number
5. Priority type
6. Observed flow range
7. Potential peak flow rate
8. Method of interception
9. Percent capture
10. Approximate capital cost for interception works
11. Level of feasibility
12. Remarks.

TABLE 2

SUMMARY OF BY-LAW VIOLATIONS

Parameter	Outfalls with By-law Violations
Phenols	284, 2, 106, 140, 741, 739, 504, 279, 383, 393, 502, 125, 123, 37, 135, 75, 85 87, 139
Zinc	502
Mercury	140

TABLE 3a. DRY WEATHER FLOW INTERCEPTION SUMMARY

MUNICIPALITY	REACH	OUTFALL NUMBER	DRAWING NO.	PRIORITY TYPE (C/B)	OBSERVED DWF RANGE (l/s)	POTENTIAL PEAK FLOW (l/s)	INTERCEPTION METHOD (1,2,3)	PERCENT CAPTURE	APPROX. CAPITAL COST $ 1000	FEASIBILITY LEVEL (L/M/H)	REMARKS
ETOBICOKE	A	284	A-1	B	0.2-0.25	13	1	100	53.28	L	check overflow for contamination source, old sanitary relief CONNECT TO METRO TRUNK
	A	298	A-2	B	0.25-3.0	9	3	100	46.68	L	sanitary sewer above storm sewer, small catchment
	A	300	A-3	B	0.10	7	2	95	5.40	H	use 90mm hydro-brake
	C	2	A-5	B	15.0	45	3	100	63.24	L	Metro trunk across river, pump to sanitary sewer on road
	C	17	A-4	B	5.5-10.0	84	2	100	66.96	L	CONNECT TO METRO TRUNK across river through golf course use 203mm hydrobrake
	C	55	-----	C	0.1-0.25	-----	-----	-----	-----	-----	insufficient data, outfall not located on any maps
	D	80	-----	C	0.1-0.25	-----	-----	-----	-----	-----	no dwgs or maps available, Metro trunk across river
	D	354	-----	B	15.0	-----	-----	-----	-----	-----	insufficient data
	E	106	A-6	B	0.05-1.0	9	1	100	63.60	L	
	G	252	A-7	B	0.1-0.25	10	1	100	53.28	L	CONNECT TO METRO TRUNK
	G	378	A-8	B	0.1	12	1	100	57.36	L	
	H	455	-----	B	1.5	-----	-----	-----	-----	-----	outfall location uncertain, Metro trunk across river
	H	465	A-9	B	1.5	17	1	100	14.28	H	
	J	140	A-10	C	0.1-2.5	13	1	100	30.84	M	

(C/B) chemical and/or bacteriological contamination
(L/M/H) low, medium or high feasibility

Interception methods 1. leaping weir 2. hydro-brake 3. sump pump

Table 3b DRY WEATHER FLOW INTERCEPTION SUMMARY

MUNICIPALITY	REACH	OUTFALL NUMBER	DRAWING NO.	PRIORITY TYPE (C/B)	OBSERVED DWF RANGE (l/s)	POTENTIAL PEAK FLOW (l/s)	METHOD OF INTERCEPTION (1,2,3)	PERCENT CAPTURE	APPROX. COST $ 1000	LEVEL OF FEASIBILITY (L/M/H)	REMARKS
NORTH YORK	G	502	A-25	C	15.0	45	3	100	37.50	M	pump to sanitary on Toryork road
	G	504	A-13	B/C	20.0	60	3	100	40.56	M	sanitary sewer lies above storm sewer
	H	441	A-24	C	2.0-2.5	12	1	100	48.36 or 42.0	M	sanitary above storm at road, CONNECT TO METRO TRUNK
	H	193	-----	B/C	10.0	-----	-----	-----	-----	-----	insufficient data
	H	209	-----	C	2.5-8.0	-----	-----	-----	-----	-----	outfall not located, no plan and profile, Metro trunk nearby
	H	213	-----	C	0.1-0.25	-----	-----	-----	-----	-----	insufficient data, outfall not located on any maps
	H	225	A-22	C	50.0	155	2	100	14.52	M	CONNECT TO METRO TRUNK, use 305 mm hydrobrake
	H	237	-----	B	1.0	-----	-----	-----	-----	-----	insufficient data, 2 catchments to one outfall
	H	247	A-15	B/C	1.5-3.0	14	2	100	23.16	M	use 102mm hydrobrake
	H	255	-----	B	1.0	-----	-----	-----	-----	-----	insufficient data, Metro trunk on other side of Black Creek
	H	269	A-14	B	0.25-0.5	17	2	100	10.56	M	CONNECT TO METRO TRUNK, use 90 mm hydrobrake
	H	739	A-12	C	5.0	30	3	100	43.68	M	sanitary sewer runs parallel to storm sewer
	H	741	A-11	C	5.0	26	2 or 3	60	10.8 or 43.2	M	storm crosses sanitary but no sanitary invert data method depends on relative invert elevations
	O	279	A-16	B/C	1.5-5.0	14	1	100	11.76	M	
	O	301	A-23	C	1.0-10.0	89	2	100	5.88	M	CONNECT TO METRO TRUNK, use 203mm hydrobrake
	O	341	A-17	B	2.0-5.0	35	2	100	14.76	M	CONNECT TO METRO TRUNK, use 153mm hydrobrake
	O	371	-----	B	0.5-2.0	-----	-----	-----	-----	-----	culvert under road, collects discharge from 3 other outfalls Metro trunk nearby
	P	383	A-18	B	3.0-8.0	87	2	100	15.24	M	use 203mm hydrobrake
	P	387	A-19	B/C	0.1-0.25	10	1	100	19.68	M	CONNECT TO METRO TRUNK
	P	393	A-20	B/C	5.0-10.0	89	2	100	18.36	M	use 203mm hydrobrake
	P	395	A-21	B	0.25-12.0	18	1	100	31.08	M	
	Q	417	-----	B	0.1	-----	-----	-----	-----	-----	insufficient data
	Q	433	-----	B	0.5-2.5	-----	-----	-----	-----	-----	insufficient data

C/B) chemical and/or bacteriological contamination
L/M/H) low, medium or high feasibility

Interception methods 1. leaping weir 2. hydro-brake 3. sump pump

TABLE 3c. DRY WEATHER FLOW INTERCEPTION SUMMARY

MUNICIPALITY	REACH	OUTFALL NUMBER	DRAWING NO.	PRIORITY TYPE (C/B)	OBSERVED DWF RANGE (l/s)	POTENTIAL PEAK FLOW (l/s)	METHOD OF INTERCEPTION (1,2,3)	PERCENT CAPTURE	APPROX. COST $ 1000	LEVEL OF FEASIBILITY (L/M/H)	REMARKS
TORONTO	A	507	A-27	B/C	10.0	30	3	100	7.80	H	intercept and control flow at adjacent pumping station
	B	509	A-26	B/C	2.5-10.0	30	3	100	70.56	L	. Metro trunk across river in Parklawn cemetery
	L	123	A-28	B/C	0.05-0.1	19	1	100	20.10	H	off MOE bacti. priority list, use 90mm hydrobrake CONNECT TO METRO TRUNK
	L	125	A-29	B/C	0.1-0.5	97	2	100	6.96	H	off MOE bacti. priority list, combine with outfall #127 CONNECT TO METRO TRUNK, 203 mm hydrobrake
	L	127	A-29	B	10.0	included in #125	2	100	included in #125	H	connect to common manhole with # 125 and CONNECT TO METRO TRUNK

(C/B) chemical and/or bacteriological contamination
(L/M/H) low, medium or high feasibility

Interception methods 1. leaping weir 2. hydro-brake 3. sump pump

TABLE 3d. DRY WEATHER FLOW INTERCEPTION SUMMARY

MUNICIPALITY	REACH	OUTFALL NUMBER	DRAWING NO.	PRIORITY TYPE (C/B)	OBSERVED DWF RANGE (l/s)	POTENTIAL PEAK FLOW (l/s)	METHOD OF INTERCEPTION (1,2,3)	PERCENT CAPTURE	APPROX. COST $ 1000	LEVEL OF FEASIBILITY (L/M/H)	REMARKS
YORK	B	4	-----	B	0.1-1.0	-----	-----	-----	-----	-----	insufficient data, Metro. storm sewer
	B	6	A-30	B	1.0	12	1	100	21.72	M	Metro trunk on other side of river
	B	10	A-31	B	0.5	1.5	3	100	72.24	M	existing storm conveys CSO connect with #12 and pump to sanitary
	B	12	A-31	B	4.0	20	3	100	included in #10	M	existing storm conveys CSO connect with #10 and pump to sanitary
	B	14	-----	C	0.1-0.25	-----	-----	-----	-----	-----	insufficient data, Metro trunk on other side of river
	B	18	-----	B/C	0.5-5.0	-----	-----	-----	-----	-----	insufficient data, Metro trunk on other side of river
	B	20	-----	B	0.07-0.1	-----	-----	-----	-----	-----	Metro storm trunk, no municpal or feasable Metro sewer to connect to
	C	28	-----	B	0.01	-----	-----	-----	-----	-----	outfall no longer in use, converted to combined sewer
	C	32	A-32	B	0.1-2.5	9	2	100	18.48	M	Metro trunk on other side of river, use 90mm hydrobrake
	D	29	A-42	C	0.25-1.5	10	2	100	20.28	M	use 90mm hydrobrake
	D	37	A-33	B	0.01-0.25	9	2	100	17.16	M	CONNECT TO METRO TRUNK to the north, use 90mm hydrobrake
	D	39	-----	C	1.5-3.0	-----	-----	-----	-----	-----	existing CSO,check for possible contamination source
	L	69	A-34	B	0.01-0.5	9	2	100	20.76	M	use 90mm hydrobrake
	L	71	-----	B	0.01-0.5	-----	-----	-----	-----	-----	old plan and profile, insufficient data
	L	75	A-36	B	0.1	.3 or 9	3 or 1	100	57.48 or 46.44	L	CONNECT TO METRO SEWER, or municipal sewer
	L	85	A-37	B	0.03-0.5	1.5 or 9	3 or 1	100	63.12 or 48.36	L	CONNECT TO METRO SEWER, or municipal sewer
	L	87	A-38	B	0.01-0.1	.3	3	100	59.88	L	sanitary above storm
	L	97	-----	B/C	5.0-7.0	-----	-----	-----	-----	-----	existing CSO
	L	109	A-39	B	1.0-10.0	80	2	100	20.28	M	2 pipes to this outfall, connect contaminated pipe CONNECT TO METRO TRUNK, use 203mm hydrobrake south of Runnymede tributary
	L	129	-----	B	5.0	-----	-----	-----	-----	-----	
	L	135	A-35	B	3.0	12	2	100	9.96	M	use 102mm hydrobrake
	L	139	A-40	B/C	2.0-5.0	26	2	100	16.56	M	use 153mm hydrobrake
	L	165	-----	B	2.0-4.33	-----	-----	-----	-----	-----	insufficient data
	M	175	A-43	C	2.0-6.0	36	2	100	14.76	H	CONNECT TO METRO TRUNK, use 153 mm hydrobrake
	M	179	A-41	B	0.1-0.5	14	1	100	30.12	M	Metro storm trunk
	M	181	A-41	B	0.25	8	2	100	37.80	M	use 90mm hydrobrake, same sanitary pipe as #179

(C/B) chemical and/or bacteriological contamination
(L/M/H) low, medium or high feasibility

Interception methods 1. leaping weir 2. hydro-brake 3. sump pump

The outfalls have been arranged in Tables 3a to 3d by Municipality, then by Reach and then by outfall number. The priority type refers to contamination assessed on Bacteriological (B) and/or Chemical (C) parameters.

Of the 68 outfalls identified as warranting abatement action and investigated in this study, 46 outfalls (68 percent) have been assessed as technically feasible for dwf interception. The breakdown of interception methods to be used are as follows:

- 12 can operate by using leaping weirs
- 20 will require the use of hydro-brakes
- 10 will require sump pumps

The four remaining sites feasible for dwf interception can operate by two choice of methods, 1 or 3, and 2 or 3.

Twenty-two (22) outfalls could not be assessed for feasibility of dwf interception, due to either total lack of available data or insufficient information on plan and profile drawings. Two of these outfalls (39, 97) have been identified as combined sewer overflows (cso). Outfall 28 no longer exists and the storm sewer is now used as a combined sewer.

The size of hydro-brake required for each site stated in terms of inlet and outlet opening is provided in the summary table, and ranges from 90 mm to 203 mm. For outfall number 225 (figure A-22) an applicable head to discharge curve was not available, and as such the size of hydro-brake and peak flows were approximated to 305 mm and 155 l/s respectively.

A more extensive use of hydro-brakes and sump pumps is required in the City of York, compared to the other municipalities. This is due to the relatively shallower sewer grades generally found in this area.

One hundred percent capture can be obtained at all but outfalls number 300 and number 741 (figures A-3 and A-11). The estimated capture for these sites is 95 and 60 percent respectively.

Three outfalls, numbers 237, 371 and 109, service more than one pipe, but the pipe that requires interception is unknown. For outfall number 109, (figure A-39) the information available on drawings indicates that either pipe can be intercepted and the cost for each will be relatively the same. Accordingly, a method and cost for one interception at that location is provided in the summary.

Fifteen (15) outfalls will require direct connection to the Metropolitan Toronto sanitary trunk sewer, being the only or most economical location for discharge of the intercepted flows. These outfalls are highlighted in Tables 3a to 3d.

The 46 feasible interceptions will collect 251.25 l/s (79 percent) of the total dwf (319.29 l/s) from the 68 outfalls investigated. These interception works will also intercept an estimated peak flow of 1,361 l/s, approximately 5.4 times the intercepted dwf. This condition, however, can only occur if all intercepting devices reach peak flows simultaneously, a highly unlikely event, considering the probability of such a storm event and the relative location of each outfall. Wet weather inflow, although restricted by either orifices, hydro-brakes or shut off switches, may pose problems in some locations. Of primary concern would be the potential for exceeding the existing sanitary sewer flow capacity. Without a detailed analysis, the volume of wet weather flow intercepted and its impact on the sanitary system cannot be properly assessed. If wet weather inflow is to be further restricted, then the use of pumps at more sites will be required. The cost for this option is presented in Section 9.

To indicate the potential load reduction feasible through dwf interception, Table 4 has been compiled. Total loadings are provided for each reach and parameter of concern in the Humber River and were obtained from the TAWMS dwf study (Ref. 1). Loadings for each outfall can be found in the Appendix. The loadings are based on either parameter mean concentrations or by concentrations assumed to be equal to the detection limit for a given parameter. These loadings represent a maximum possible loading based on the data collected in the TAWMS study.

TABLE 4. DRY WEATHER FLOW LOADINGS
FROM FEASIBLE DWF INTERCEPTION SITES

REACH	NUMBER OF OUTFALLS	AVERAGE CONTAMINANT LOADINGS *** (grams/day)							
		Cr	Cu	Fe	Pb	Zn	Hg	PHENOLS*	FC**
A	4	58.45	19.46	253.28	78.00	76.08	0.00	0.02	16736
B	4	60.90	106.70	1059.90	81.20	28.80	0.00	0.03	15556
C	3	202.50	67.70	1582.40	164.56	42.70	0.00	1.32	95719
D	2	33.45	1.41	2.98	5.08	8.71	0.00	0.03	56400
E	1	2.64	1.10	3.75	3.53	1.10	0.95	0.05	52571
G	4	2939.06	220.85	17496.80	849.01	8356.82	0.00	114.21	44455
H	2	15.57	7.45	33.05	25.87	17.53	0.00	0.04	2693
J	1	66.25	3.47	681.80	9.24	29.28	28.31	0.00	90
L	10	115.72	51.54	2113.57	158.33	72.69	0.86	8.20	2510662
M	3	23.75	7.91	46.57	31.66	12.94	0.00	0.00	124489
N	5	297.82	146.79	7743.56	558.75	317.07	0.00	47.08	177415
O	3	67.13	22.38	559.44	98.58	107.51	0.00	42.43	4385
P	4	101.63	33.87	381.58	161.37	129.82	21.55	2.50	45033
TOTAL	46	3984.87	690.63	31958.68	2225.18	9201.05	51.67	215.91	3146204

* Units in mg/day
** Units in counts/100ml
*** DATA FROM REF. 1.

Cr - Chromium   Cu - Copper   Fe - Iron   Pb - Lead
Zn - Zinc   Hg - Mercury   Phenols - Phenolic compounds
FC - Fecal Coliform

8 - SPECIAL CONSIDERATIONS

In addition to the considerations outlined above, the following points which may be of interest with respect to installation of the interception works are presented:

1. Intercepted portions of wet weather flow allow for treatment of the initial surface washoff, potentially the most contaminated portion of stormwater runoff.

2. Installation of dwf interceptors can also provide a margin of protection against the spill of hazardous contaminants, of particular interest in areas of high sensitivity or where potential for spills are greatest.

3. Incorporating a manhole at the connection to the sanitary sewer may be desirable to allow access for further monitoring of intercepted dwf.

4. The use of water traps to prevent sanitary sewer gases from entering the storm sewer may be desirable. Water traps rarely dry up due to the sewer environment and are self generated by hydraulic design.

5. Installation of small tanks, either inline with the sanitary sewer or with interception works, may be desirable in detaining and controlling the rate of wet weather flow.

9 - ESTIMATED COSTS FOR INTERCEPTION WORKS AND TREATMENT

An estimate of costs has been prepared for the work required for each feasible site, based on 1984 construction costs. Costs include for manholes, piping, intercepting devices, installation, restoration, excavation, mobilization of crews, orifice restrictors, modifications to existing manholes and sewers and an allowance of 20 percent for engineering and contingencies. A summary of the estimated capital costs for each site is shown in Tables 3a - 3d. Basic unit costs used for the estimate are shown in Table 5. The total estimated capital cost for all feasible sites is $1.42 million.

The estimated annual treatment cost is $357,000, based on a treatment cost of $0.0449 per cubic metre (Ref. 4) for sewage receiving both primary and secondary treatment, and a total estimated intercepted dwf of 251 l/s. Costs for interception work and treatment are presented by reach in Table 6.

Additional costs will also be incurred for operation and maintenance of the works, including power and replacement of equipment subject to wear. As an average annual cost for operation and maintenance we suggest an allowance of $70,000 be made, resulting in a total annual cost of about $427,000. Similar to the cost estimate prepared for Tasks 1 to 5 inclusive, the annual costs have been converted to capital costs, using a 70 year time period and a net average interest rate (interest rate minus inflation rate) of 7 percent, equal to $5.97 million.

In order to reduce the potential inflow rate, the use of pumps will be required at more sites. If all sites were to use sump pumps the estimated capital cost is $2.5 million, an increase of $1.08 million. Average annual operation and maintenance cost would increase to $260,000. This, however, may be offset by reduced costs for treatment and relief works.

TABLE 5

BASIC UNIT COSTS

Item	Unit Cost
Hydro-brake	$150 per 25 mm dia.
Weir	$300/unit
Manholes with pumps	$28,000/unit *
Tunneling	$1,000/m **
Restrictors	$200/unit
Piping	$560/m

NOTE: All costs include installation
* Cost is the mean cost for feasible sites

** Required for outfalls numbers 17, 75 and 85 (Figures A-4, A-36 and A-37)

TABLE 6

WORKS AND TREATMENT COSTS BY REACH

Reach	No. of feasible sites	dwf intercepted (l/s)	Works cost ($1000)	Annual treatment cost ($1000)
A	4	13.35	113	19
B	4	15.5	165	22
C	3	27.5	149	39
D	2	1.75	38	2
E	1	1.1	84	2
G	4	35.35	189	50
H	2	4.0	63	6
J	1	2.5	31	4
L	10	29.7	255	42
M	3	6.75	83	10
N	5	63.5	135	90
O	3	20.0	32	28
P	4	30.25	84	43
Q	0	0.0	0	0
Total	46	251.25	1,421	357

10 - SUMMARY AND CONCLUSIONS

Recent studies for the Ministry of the Environment of dry weather discharges from sewer outfalls to the Humber River and its tributaries have identified 68 outfalls as requiring abatement action, due to their relative levels of contamination. The purpose of this study was to investigate the feasibility of intercepting these discharges into the sanitary sewer system,and to provide estimated costs for interception works and treatment.

The study methodology established alternative methods for dwf interception that would meet the objectives of the study. Three methods were found suitable for the range of conditions found; leaping weirs, hydro-brakes and sump pumps.

Of the 68 outfalls investigated, twenty-two outfalls were not assessed due to a lack of available data or insufficient information, and 46 outfalls (68 percent) have been assessed as technically feasible for dwf interception. The breakdown of interception methods to be used are as follows:

- 12 can operate by using leaping weirs
- 20 will require the use of hydro-brakes
- 10 will require sump pumps

The four remaining sites feasible for dwf interception can operate by a choice of methods 1 or 3, and 2 or 3.

Fifteen outfalls will require direct connection to the Metropolitan Toronto sanitary trunk sewer.

The 46 feasible interceptions will collect 251.25 l/s of dwf, or 79 percent of the total dwf for the 68 outfalls. The estimated capital costs for the installation of interception works is $1.42 million. An annual cost of $427,000 is required for operation, maintenance and treatment. Capitalizing the annual costs over a 70 year period results in an equivalent capital cost of $5.97 million.

Without a complex detailed analysis, the volume of wet weather flow intercepted and the impact on the existing sanitary sewer system can not be assessed. However, the peak inflow rate was estimated to be 1,361 l/s, 5.4 times the intercepted dwf rate. If pumps were used at all sites to reduce the potential wet weather inflow rate, the estimated works cost would increase to $2.5 million and annual operation and maintenance costs to $260,000.

Under existing bylaws, the interception of dwf from storm to sanitary sewer is not feasible (except for the City of Toronto). This is primarily based on the premise that storm water is uncontaminated and not in need of treatment. Secondly, all municipalities have concerns with respect to the available flow capacities within their existing sanitary or combined sewer systems and the potential for basement flooding. If dwf interception is to be carried out as presented, revision or amendment to the existing sewer-use bylaws is required.

Nineteen of the feasible sites were found to have at least one sample in violation of the Metropolitan Toronto sanitary sewer use by-law for phenolic compounds. One outfall had a single violation of mercury and another of zinc.

REFERENCES

1. Toronto Area Watershed Management Strategy Study. Technical Report #1, Humber River and Tributary Dry Weather Outfall Study. November 1983.

2. Babbit, Harold E. Sewer and Sewage Treatment, Seventh Edition. 1953.

3. The Municipality of Metropolitan Toronto By-law No. 148-83. To Regulate the Discharge of Sewage and Land Drainage in the Metropolitan Area.

4. Ministry of the Environment. Correspondence, Memorandum. August 21, 1985.

APPENDIX

Outfall Loadings

Drawings of Proposed Interception Locations and Schemes

DRY WEATHER FLOW LOADINGS
FROM FEASIBLE DWF INTERCEPTION SITES

AVERAGE CONTAMINANT LOADINGS ***
(grams/day)

REACH	OUTFALL NUMBER	Cr	Cu	Fe	Pb	Zn	Hg	PENOLS*	FC**
A	284	1.21	0.39	30.03	1.61	1.68		0.02	315
	298	4.92	1.60	75.50	6.60	4.10			162
	300	0.52	0.17	9.55	0.69	0.30			15649
	507	51.80	17.30	138.20	69.10	760.00			610
REACH A TOTAL		58.45	19.46	253.28	78.00	766.08	0.00	0.02	16736
B	6	5.20	1.70	12.10	6.90	13.00		0.03	8916
	10	2.60	0.90	8.00	3.50	1.90			2300
	12	20.70	6.90	13.80	27.60	6.90			2291
	509	32.40	97.20	1026.00	43.20	7.00			2049
REACH B TOTAL		60.90	106.70	1059.90	81.20	28.80	0.00	0.03	15556
C	2	155.50	52.00	1205.00	104.00	26.00		1.30	1102
	17	41.00	13.70	355.00	54.60	13.70			81
	32	6.00	2.00	22.40	5.96	3.00		0.02	94536
REACH C TOTAL		202.50	67.70	1582.40	164.56	42.70	0.00	1.32	95719
D	29	32.57	1.12	2.25	3.90	8.42	0.95		77
	37	0.88	0.29	0.73	1.18	0.29		0.03	56323
REACH D TOTAL		33.45	1.41	2.98	5.08	8.71	0.95	0.03	56400
E	106	2.64	1.10	3.75	3.53	1.10		0.05	52571
REACH E TOTAL		2.64	1.10	3.75	3.53	1.10	0.00	0.05	52571
G	252	1.14	0.38	0.76	1.52	1.90			11335
	378	0.52	0.17	0.03	0.69	0.02			25210
	502	2851.00	116.60	13349.00	466.60	7905.60		35.09	110
	504	86.40	103.70	4147.00	380.20	449.30		79.12	7800
REACH G TOTAL		2939.06	220.85	17496.80	849.01	8356.82	0.00	114.21	44455

* Units in mg/day
** Units in counts/100ml
*** DATA FROM REF. 1.

Cr - Chromium Cu - Copper Fe - Iron Pb - Lead
Zn - Zinc Hg - Mercury Phenols - Phenolic compounds
FC - Fecal Coliform

DRY WEATHER FLOW LOADINGS
FROM FEASIBLE DWF INTERCEPTION SITES

REACH	OUTFALL NUMBER	AVERAGE CONTAMINANT LOADINGS *** (grams/day)							
		Cr	Cu	Fe	Pb	Zn	Hg	PHENOLS*	FC**
H	465	7.78	2.59	7.78	10.37	9.72			2683
	441	7.79	4.86	25.27	15.50	7.81		0.04	10
REACH H TOTAL		15.57	7.45	33.05	25.87	17.53	0.00	0.04	2693
J	140	66.25	3.47	681.80	9.24	29.28	28.31		90
REACH J TOTAL		66.25	3.47	681.80	9.24	29.28	28.31	0.00	90
L	69	2.08	2.43	16.00	2.78	2.43			516769
	75	0.52	0.22	0.56	0.69	0.22		0.32	25944
	85	0.26	0.26	3.68	1.05	1.31		0.18	136302
	87	0.43	0.81	2.41	0.57	0.35		0.06	74632
	109	25.92	17.28	75.60	34.56	15.12			5446
	123	0.55	1.40	71.60	1.70	7.92		3.22	500219
	125	1.01	0.78	56.76	3.62	4.04	0.86	2.15	1235611
	127	51.80	17.30	976.30	69.12	30.24		0.60	759
	135	15.55	5.18	44.06	20.74	5.18		0.83	14748
	139	17.60	5.88	866.60	23.50	5.88		0.84	232
REACH L TOTAL		115.72	51.54	2113.57	158.33	72.69	0.86	8.20	2510662
M	175	20.74	6.91	13.82	27.65	10.37			1700
	179	1.71	0.57	27.35	2.28	2.14			
	181	1.30	0.43	5.40	1.73	0.43			122789
REACH M TOTAL		23.75	7.91	46.57	31.66	12.94	0.00	0.00	124489
N	225	259.20	86.40	367.20	345.60	108.00			2200
	247	10.89	3.63	3204.23	16.33	13.61			6830
	269	1.81	0.60	3.33	2.42	1.06			168365
	739	17.28	30.24	2980.80	125.28	103.68		20.30	10
	741	8.64	25.92	1188.00	69.12	90.72		26.78	10
REACH N TOTAL		297.82	146.79	7743.56	558.75	317.07	0.00	47.08	177415

* Units in mg/day
** Units in counts/100ml
*** DATA FROM REF. 1.

Cr - Chromium Cu - Copper Fe - Iron Pb - Lead
Zn - Zinc Hg - Mercury Phenols - Phenolic compounds
FC - Fecal Coliform

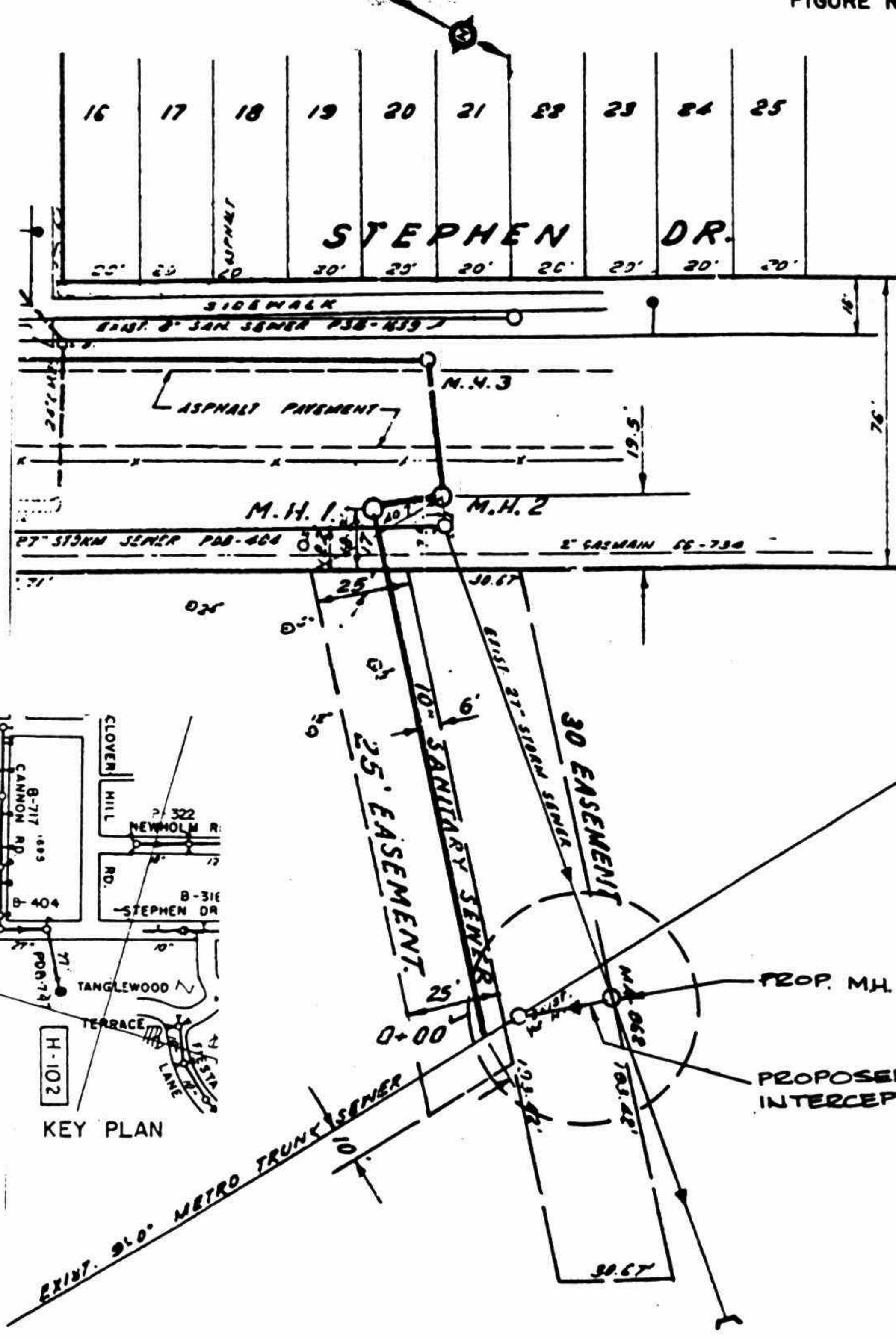
DRY WEATHER FLOW LOADINGS
FROM FEASIBLE DWF INTERCEPTION SITES

REACH	OUTFALL NUMBER	AVERAGE CONTAMINANT LOADINGS *** (grams/day)							
		Cr	Cu	Fe	Pb	Zn	Hg	PHENOLS*	FC**
O	279	21.77	7.26	281.23	38.10	59.88		42.37	266
	301	27.22	9.07	163.30	36.29	34.02			42
	341	18.14	6.05	114.91	24.19	13.61		0.06	4077
REACH O TOTAL		67.13	22.38	559.44	98.58	107.51	0.00	42.43	4385
P	383	29.03	9.68	142.73	38.71	33.87		1.84	3150
	387	0.67	0.22	2.19	0.90	7.64			4113
	393	36.94	12.31	198.75	75.10	47.49	21.55	0.54	26714
	395	34.99	11.66	37.91	46.66	40.82		0.12	11056
REACH P TOTAL		101.63	33.87	381.58	161.37	129.82	21.55	2.50	45033

* Units in mg/day
** Units in counts/100ml
*** DATA FROM REF. 1.

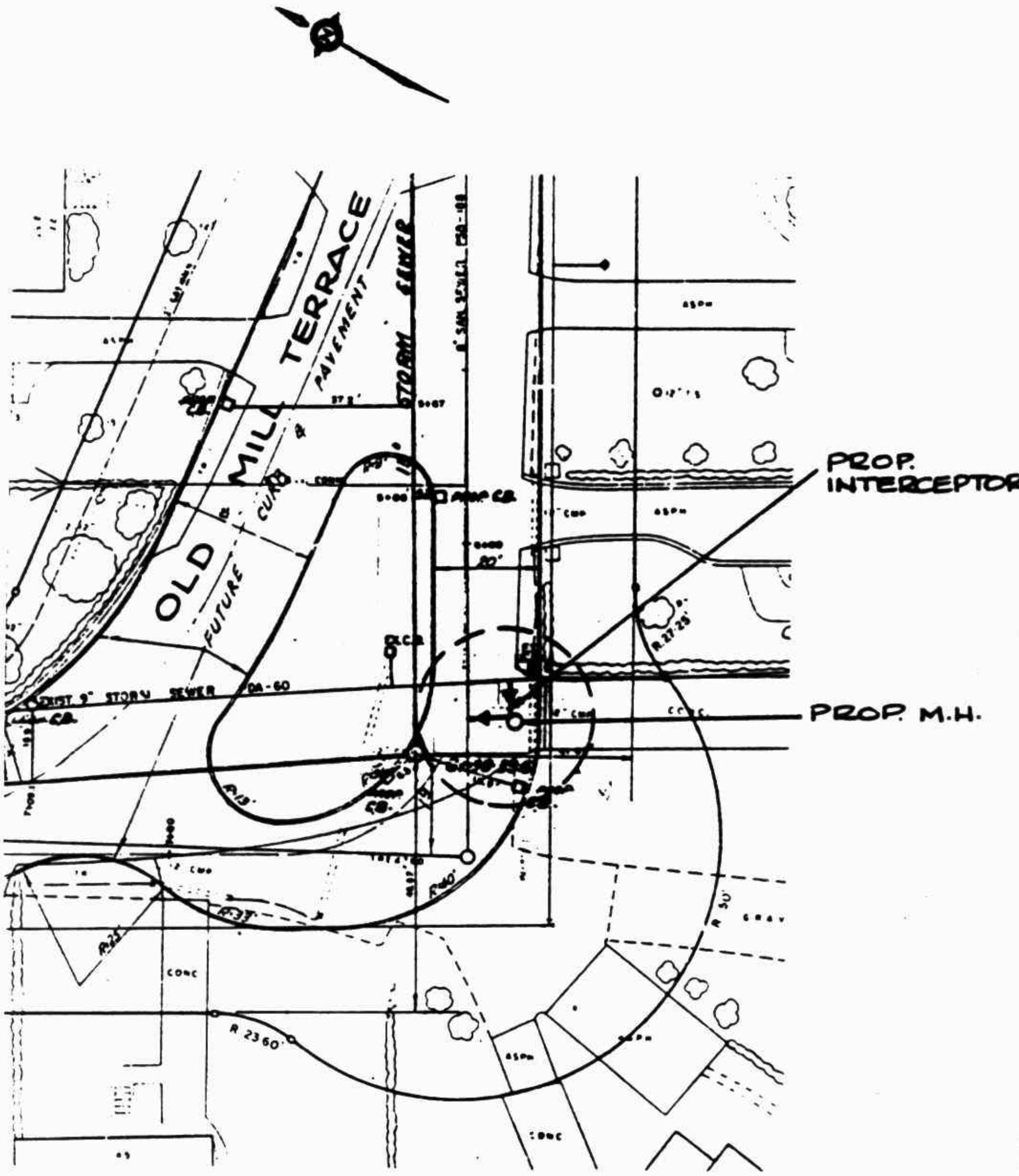
Cr - Chromium Cu - Copper Fe - Iron Pb - Lead
Zn - Zinc Hg - Mercury Phenols - Phenolic compounds
FC - Fecal Coliform

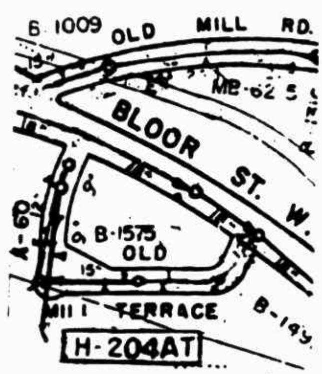
FIGURE No: A-1



	INVERT (m.)	GRADE (%)	SIZE (mm.)	LENGTH (m.)	METHOD: 1	OUTFALL No.
STORM	84·36	—	675	—		
SANITARY	79·96	2·27	2700	—	SCALE: 1:500	A - 284
INTERCEPTOR	—	—	100	9·0		

FIGURE No: A-2





KEY PLAN

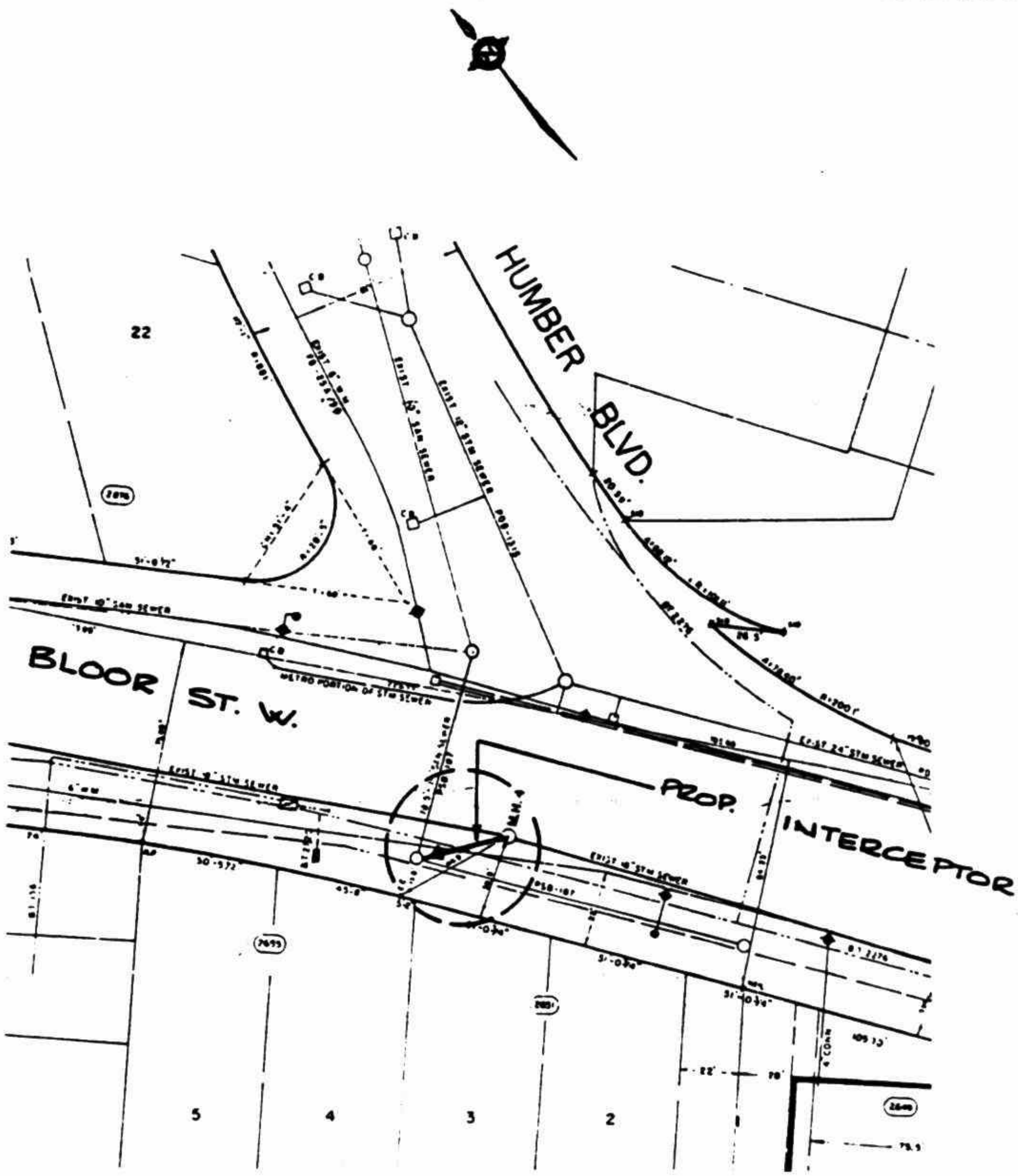
	INVERT (m.)	GRADE (%)	SIZE (mm.)	LENGTH (m.)
STORM	96.31	6.24	225	—
SANITARY	98.14	2.20	200	—
INTERCEPTOR	—	—	100	10

METHOD: 3

SCALE: 1:500

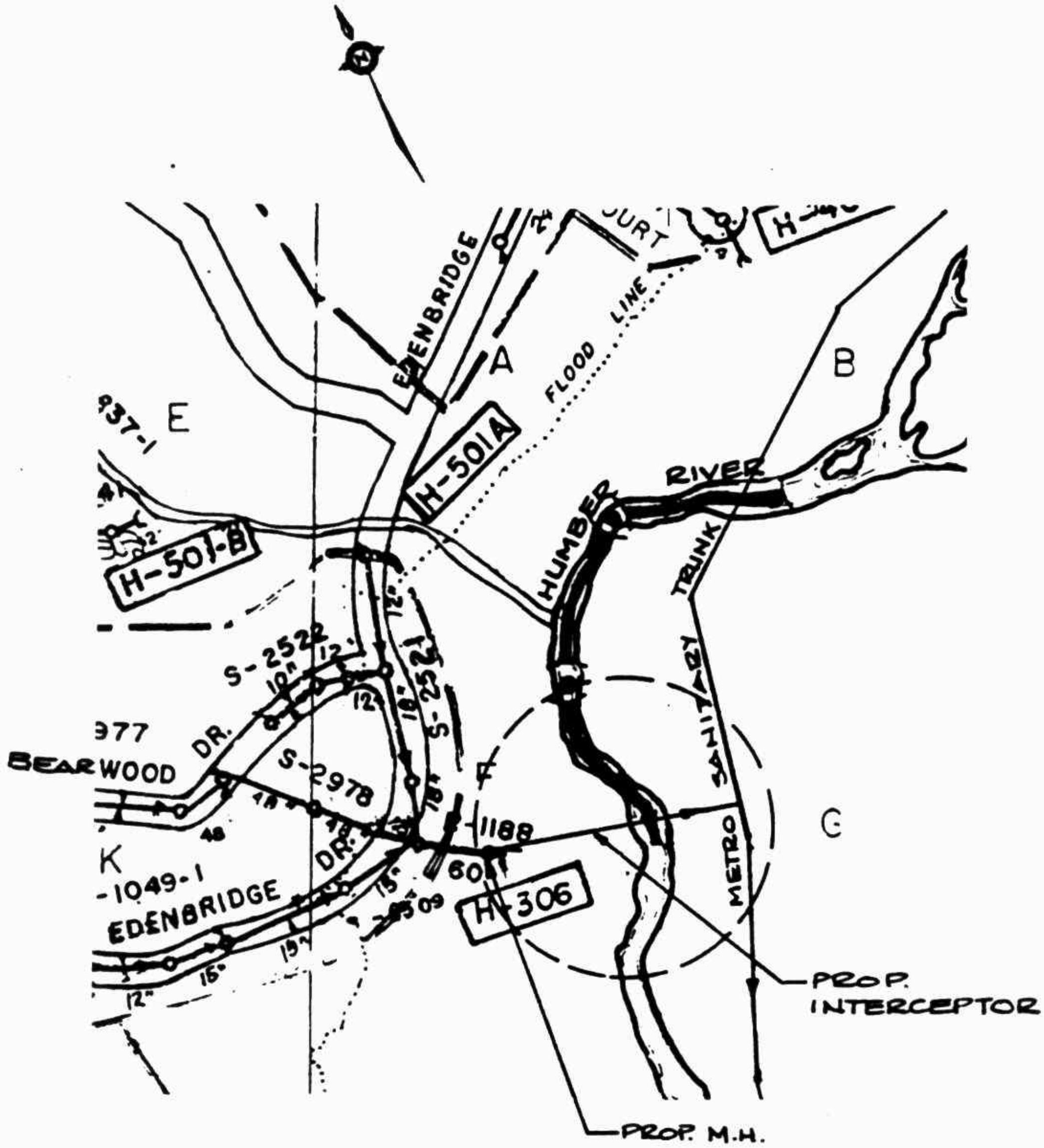
OUTFALL No. A-298

FIGURE No: A-3



	INVERT (m.)	GRADE (%)	SIZE (mm.)	LENGTH (m.)	METHOD: 2	OUTFALL No.
STORM	92·35	4·12	450	—		
SANITARY	90·83	—	250	—	SCALE: 1:500	A-300
INTERCEPTOR	—	—	100	9·1		

FIGURE No: A- 4



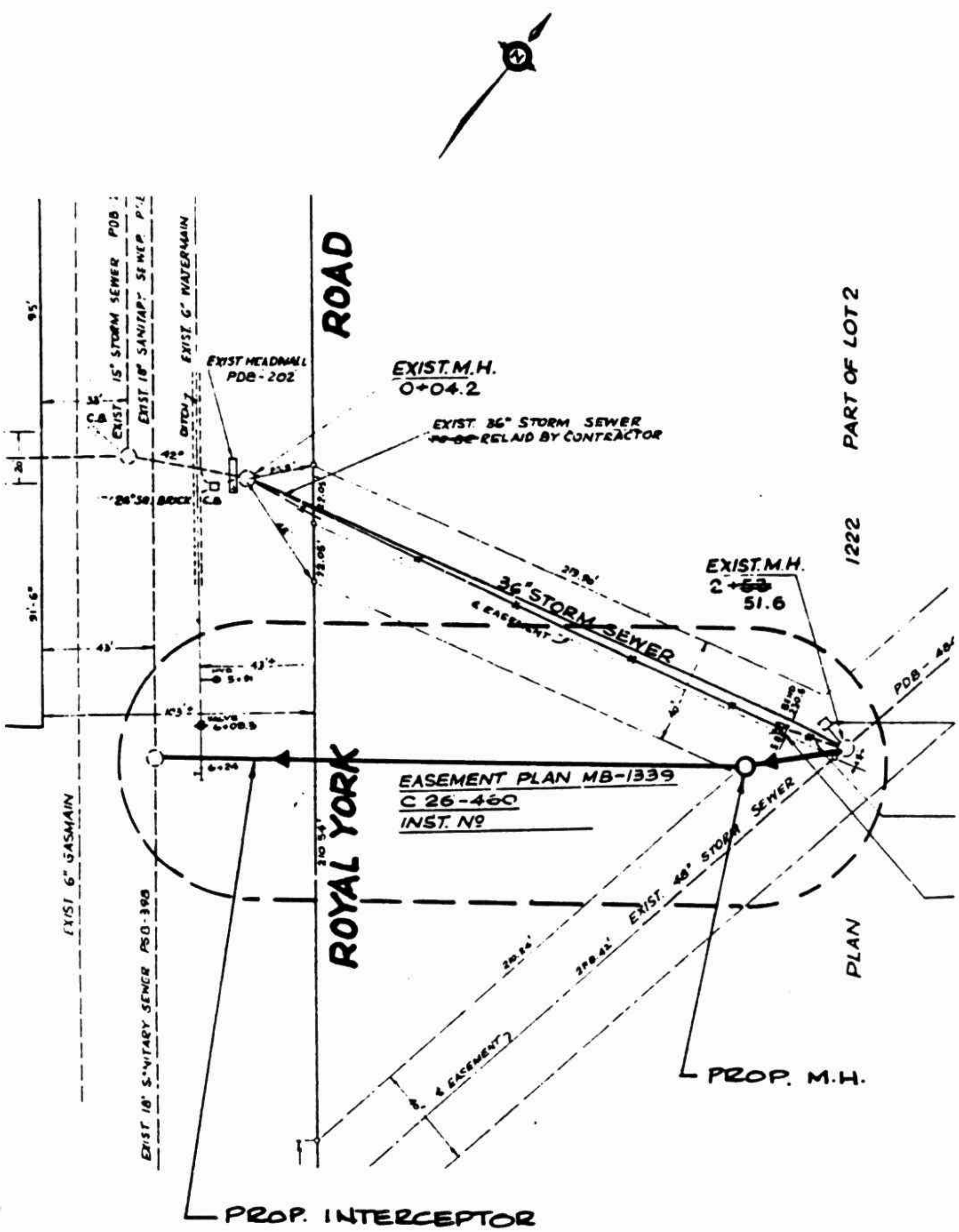
	INVERT (m.)	GRADE (%)	SIZE (mm.)	LENGTH (m.)
STORM	—	—	1500	—
SANITARY	—	—	1800	—
INTERCEPTOR	—	—	200	152

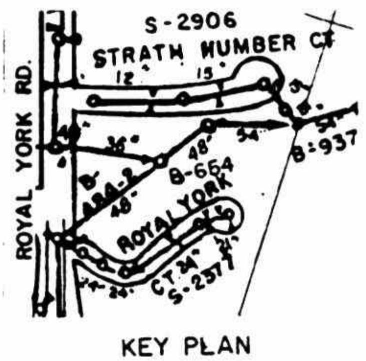
METHOD: 2

SCALE: 1:5000

OUTFALL No. C - 17

FIGURE No: A-5

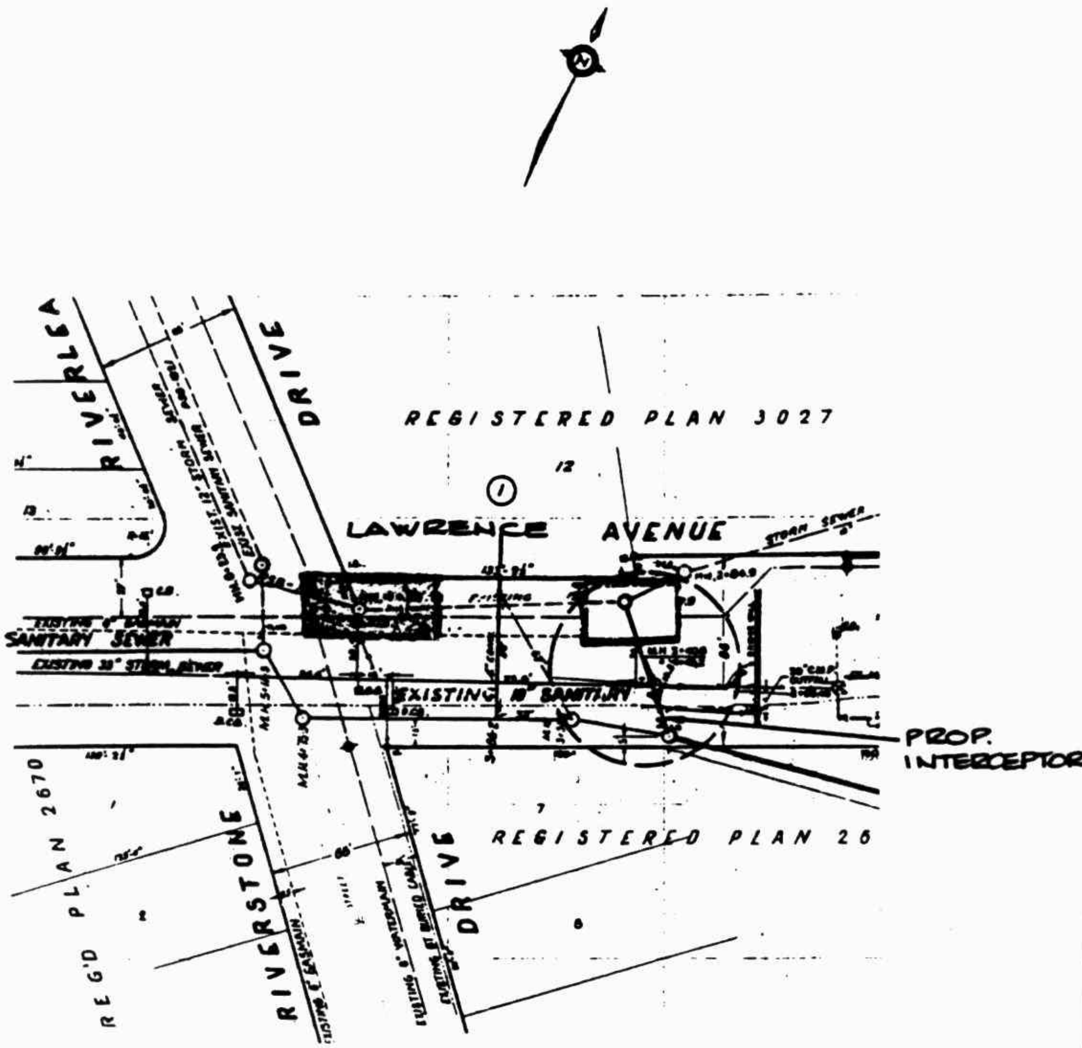




KEY PLAN

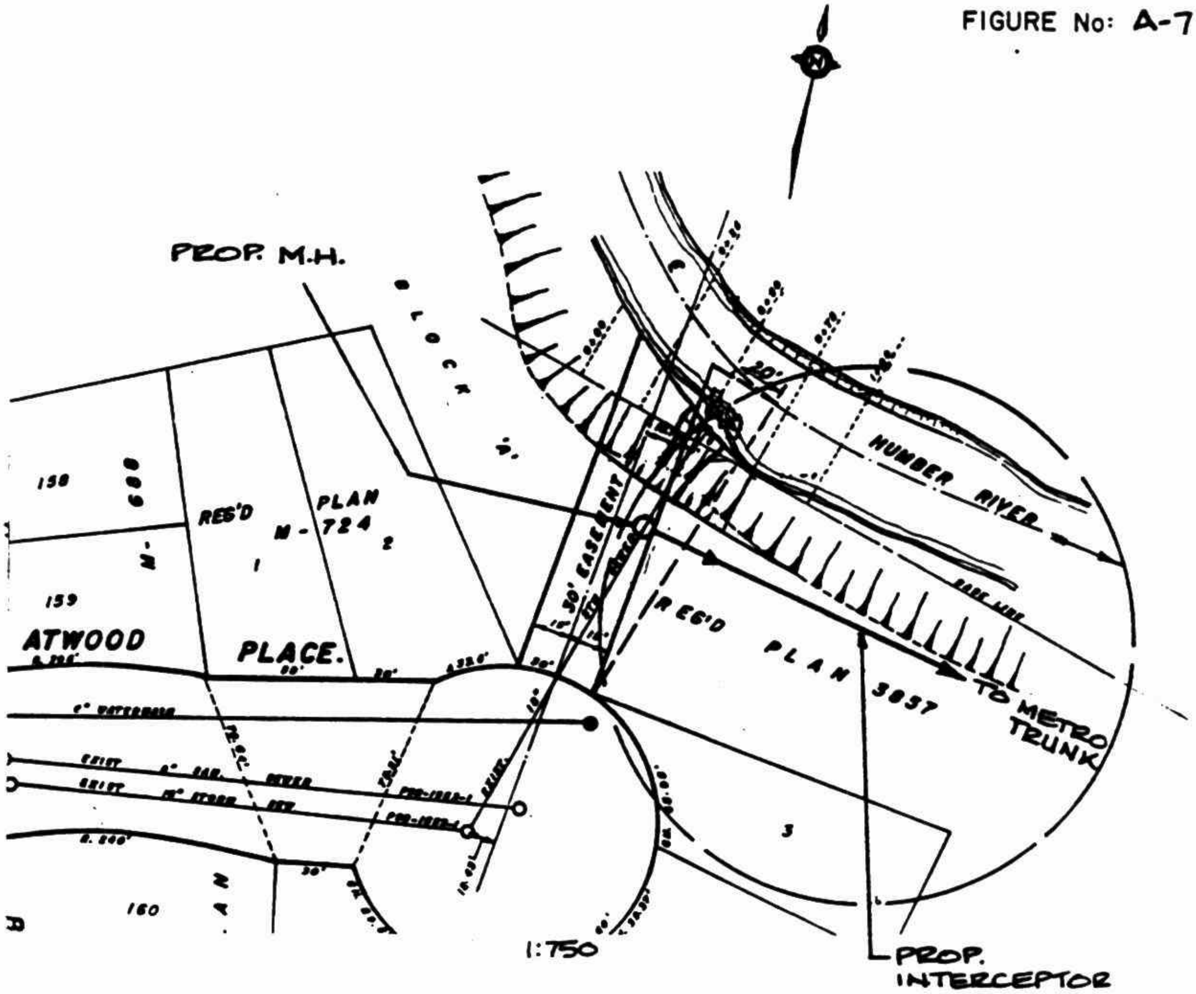
	INVERT (m.)	GRADE (%)	SIZE (mm.)	LENGTH (m.)	METHOD: 3	OUTFALL No.
STORM	110.96	4.46	1350	—		C-2
SANITARY	—	—	450	—	SCALE: 1:750	
INTERCEPTOR	—	—	100	66.9		

FIGURE No: A-6



	INVERT (m.)	GRADE (%)	SIZE (mm.)	LENGTH (m.)	METHOD: 1	OUTFALL No.
STORM	118·26	39	250	—		
SANITARY	115·51	15	250	—	SCALE: 1:1000	E-106
INTERCEPTOR	—	—	100	20		

FIGURE No: A-7



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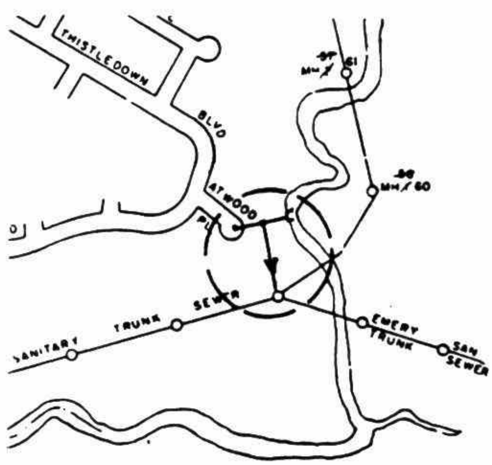
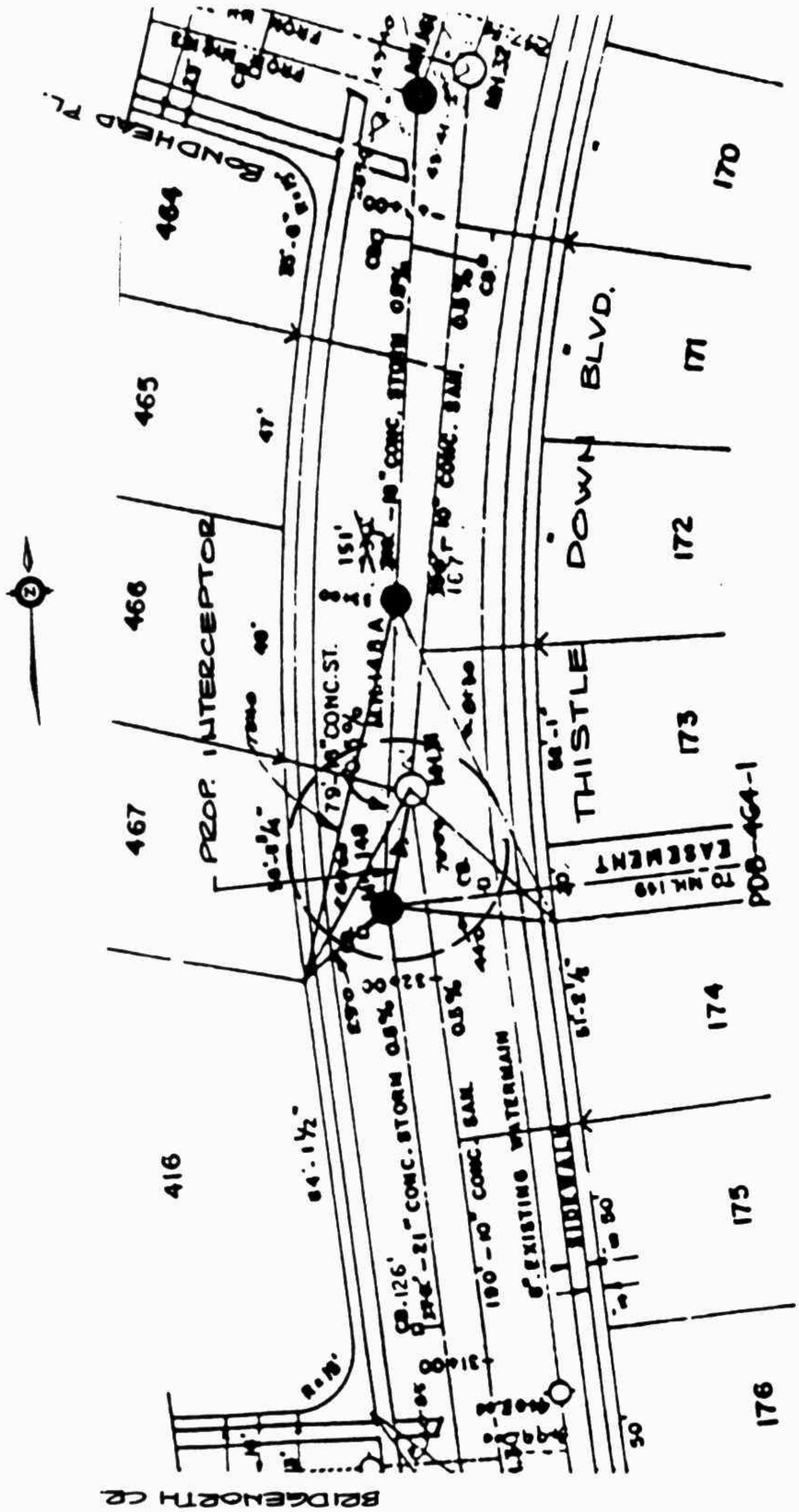


1:5000

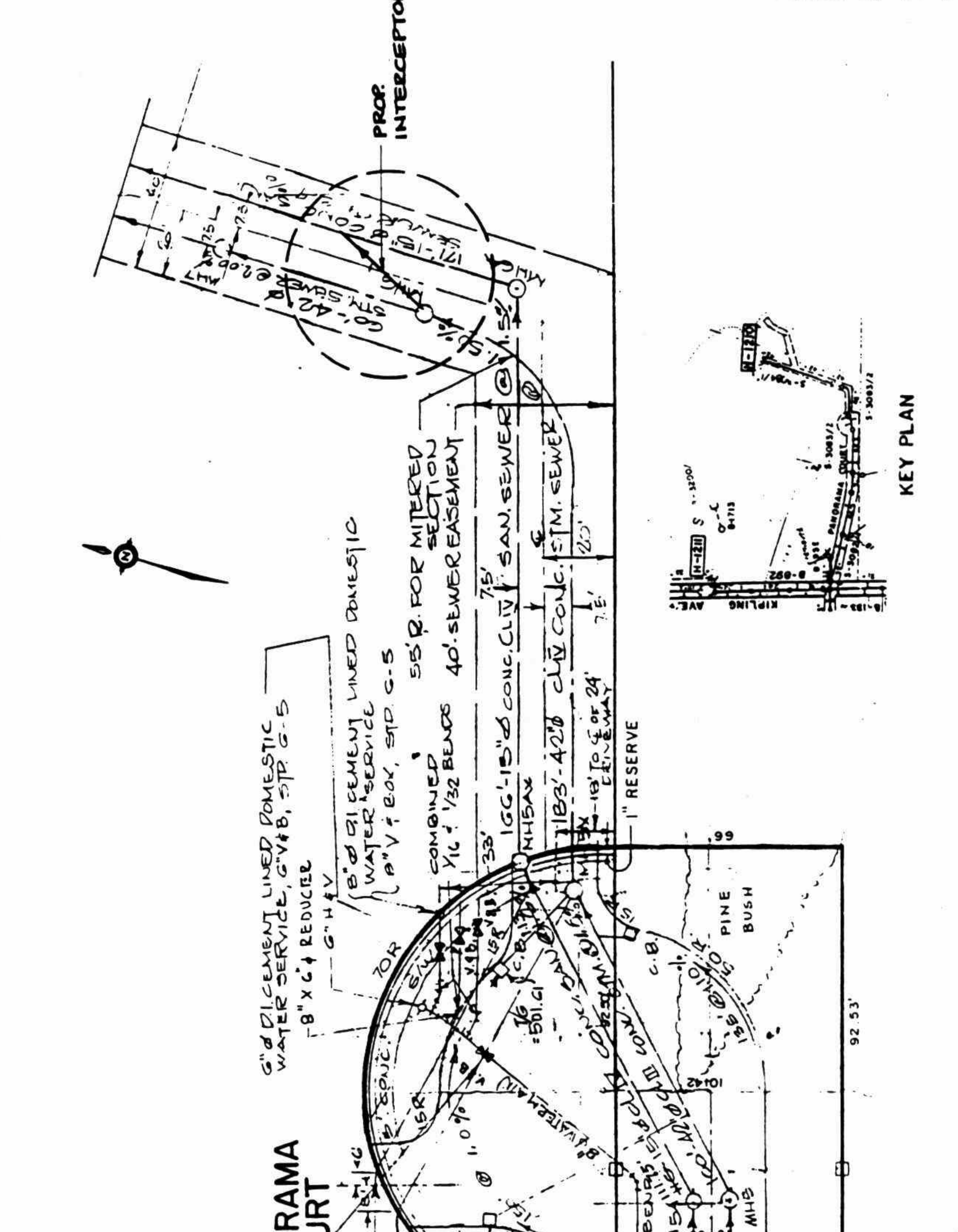
	INVERT (m.)	GRADE (%)	SIZE (mm.)	LENGTH (m.)	METHOD: 1	OUTFALL No.
STORM	128·60	8·2	375	—		G-252
SANITARY	—	—	—	—	SCALE: AS NOTED	
INTERCEPTOR	—	—	100	114		

FIGURE No: A-8



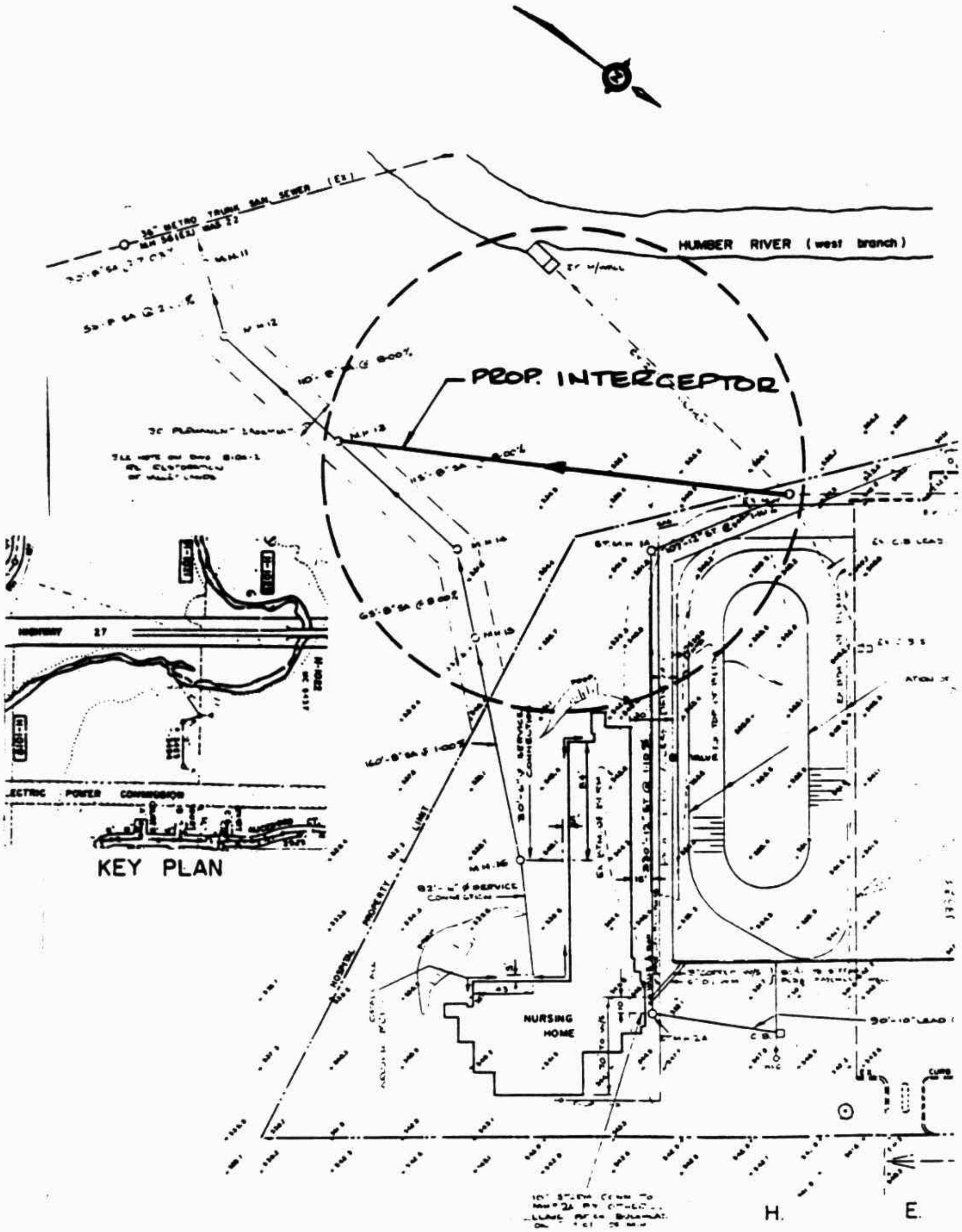
	INVERT (m.)	GRADE (%)	SIZE (mm.)	LENGTH (m.)	METHOD: 1	OUTFALL No.
STORM	129·34	1·94	450	—		
SANITARY	128·89	0.50	250	—	SCALE: 1:500	G-378
INTERCEPTOR	—	—	100	8·51		

FIGURE No: A-9



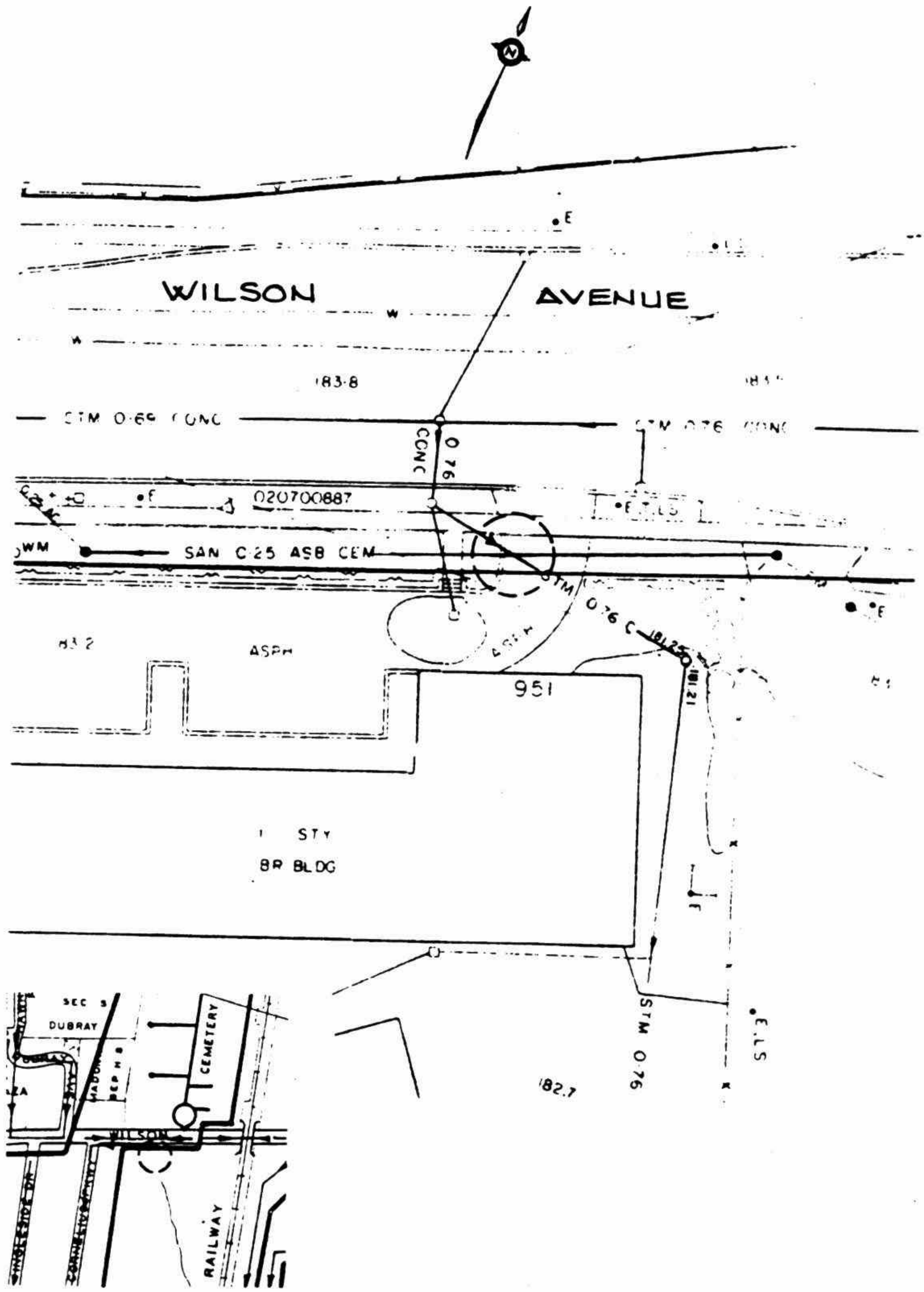
	INVERT (m.)	GRADE (%)	SIZE (mm.)	LENGTH (m.)	METHOD: 1	OUTFALL No.
STORM	147.11	2.00	1050	–		H-465
SANITARY	145.32	2.70	375	–	SCALE: 1:500	
INTERCEPTOR	–	–	100	9.1		

FIGURE No: A-10



	INVERT (m.)	GRADE (%)	SIZE (mm.)	LENGTH (m.)	METHOD: 1	OUTFALL No.
STORM	158.95	—	600	—		J-140
SANITARY	151.94	7.54	200	—	SCALE: 1:1500	
INTERCEPTOR	—	—	100	100		

FIGURE No: A-11

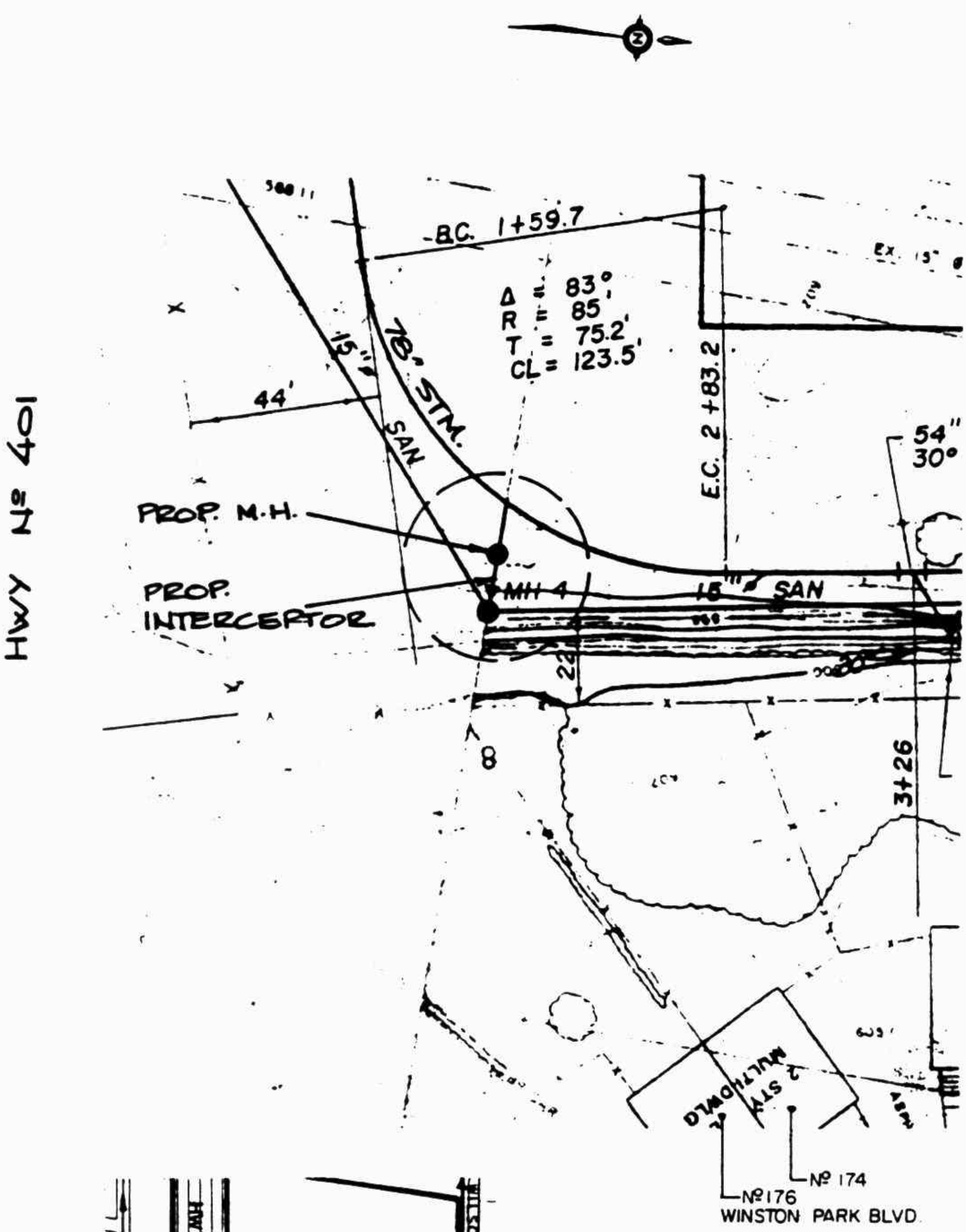


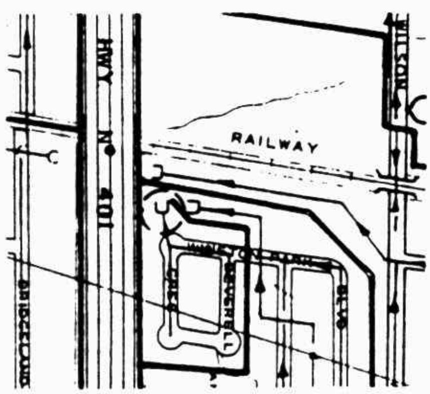
KEY PLAN

	INVERT (m.)	GRADE (%)	SIZE (mm.)	LENGTH (m.)	METHOD: 2	OUTFALL No.
STORM	—	—	760	—		N-741
SANITARY	—	—	250	—	SCALE: 1:500	
INTERCEPTOR	—	—	150	1·0		

	INVERT (m.)	GRADE (%)	SIZE (mm.)	LENGTH (m.)	METHOD: 3	OUTFALL No.
STORM	—	—	760	—		N-741
SANITARY	—	—	250	—	SCALE: 1:500	
INTERCEPTOR	—	—	100	1·8		

FIGURE No: A-12

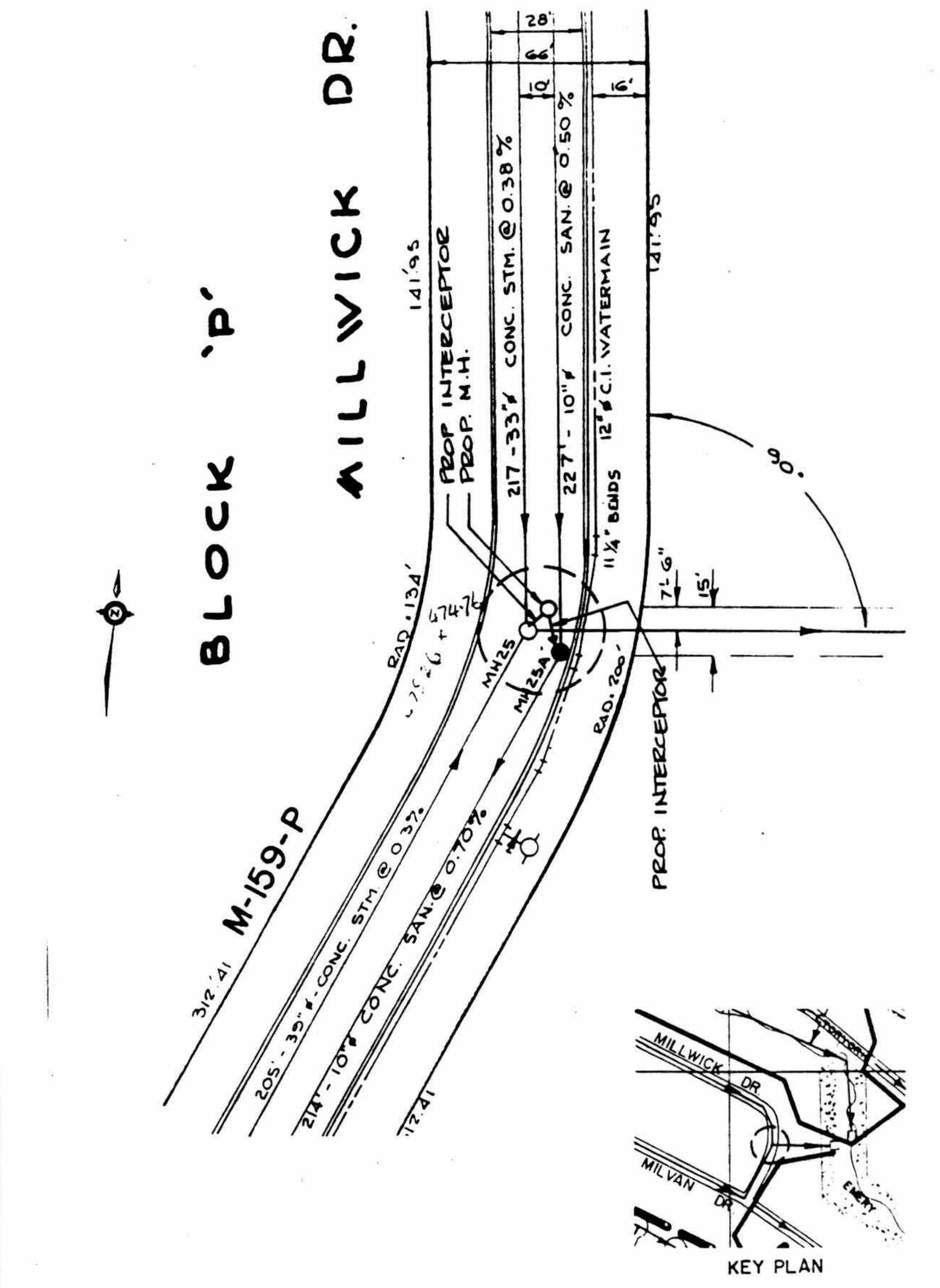




KEY PLAN

	INVERT (m.)	GRADE (%)	SIZE (mm.)	LENGTH (m.)	METHOD: 3	OUTFALL No.
STORM	179.06	0.4	1950	—		N-739
SANITARY	179.36	0.6	375	—	SCALE: 1:500	
INTERCEPTOR	—	—	100	6.1		

FIGURE No: A-13



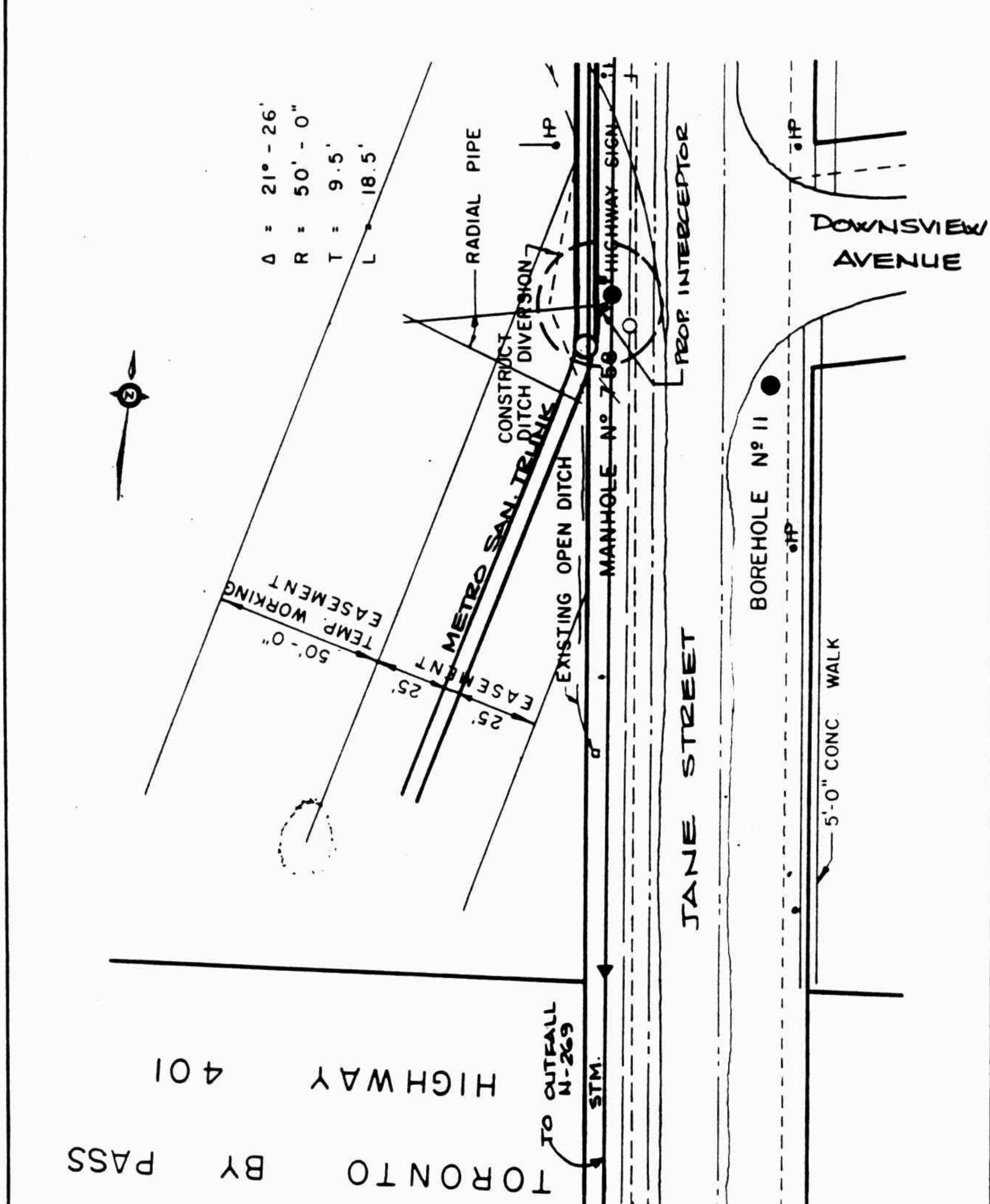
	INVERT (m.)	GRADE (%)	SIZE (mm.)	LENGTH (m.)
STORM	146.20	0.6	1050	—
SANITARY	147.85	0.7	250	—
INTERCEPTOR	—	—	150	3.1

METHOD: 3

SCALE: 1:500

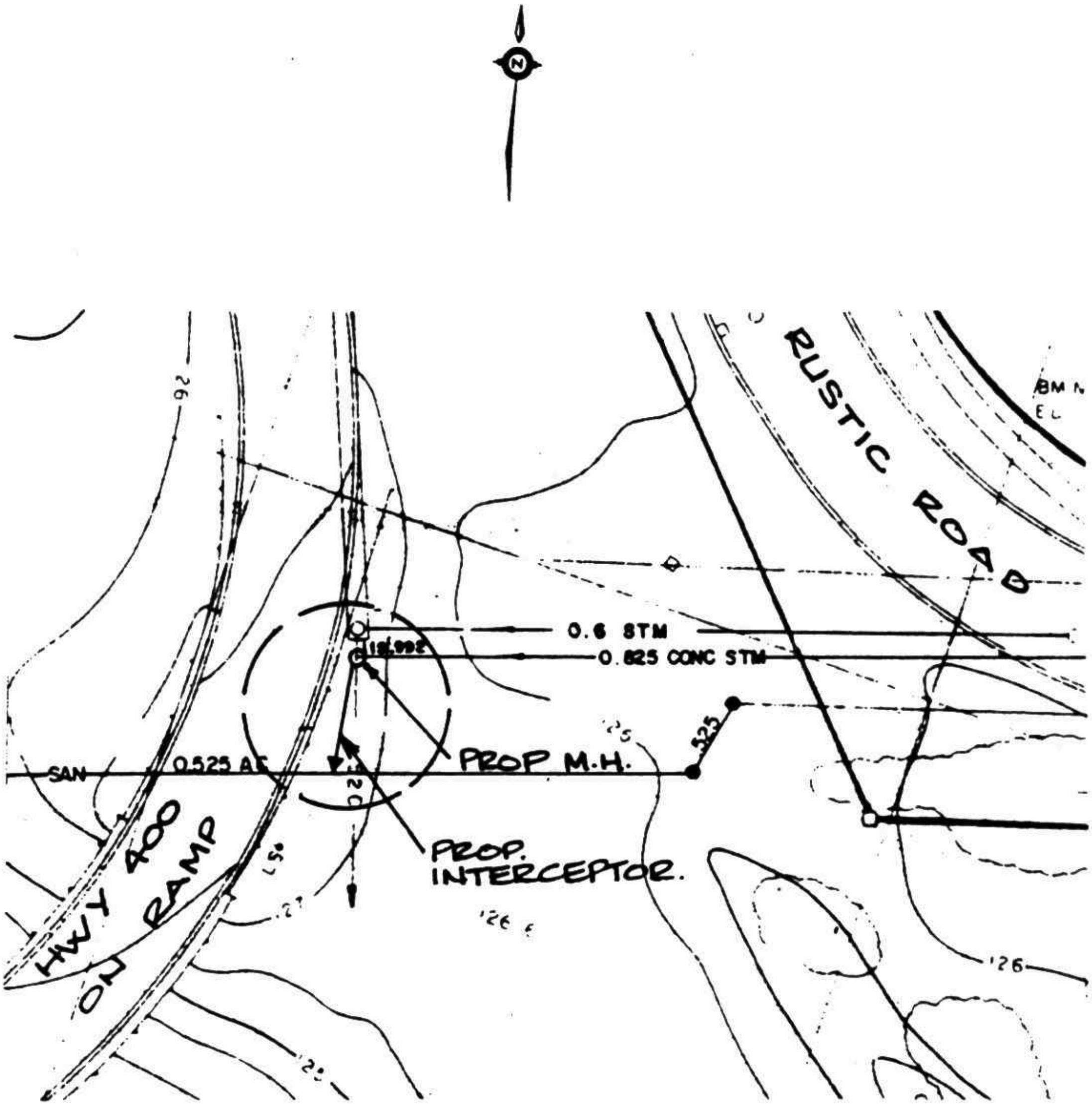
OUTFALL No. G-504

FIGURE No: Δ-14



	INVERT (m.)	GRADE (%)	SIZE (mm.)	LENGTH (m.)	METHOD: 2	OUTFALL No.
STORM	121.65	—	1050	—		N-269
SANITARY	118.65	0.30	1200	—	SCALE: 1:500	
INTERCEPTOR	—	—	100	3.0		

FIGURE No: A-15



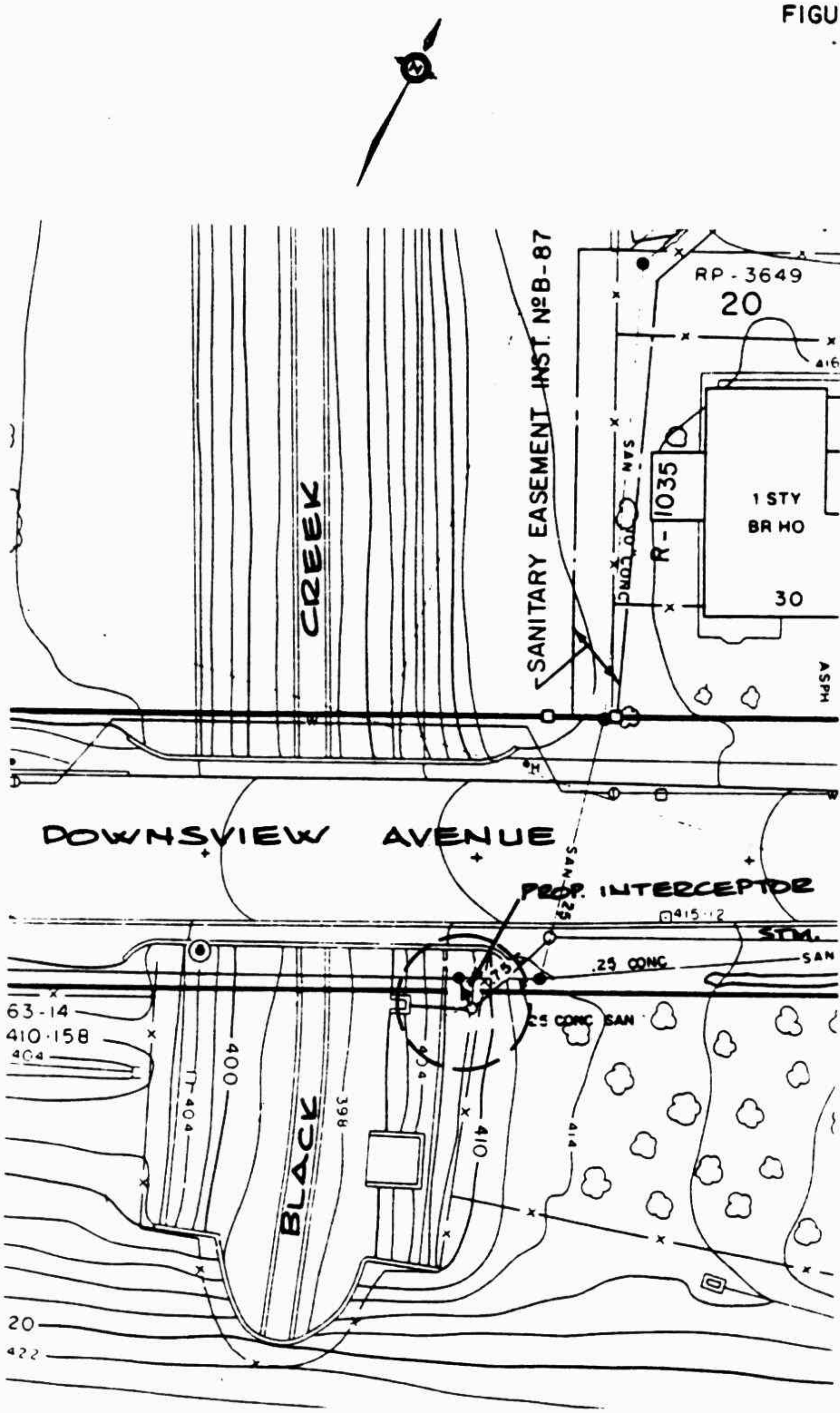
	INVERT (m.)	GRADE (%)	SIZE (mm.)	LENGTH (m.)
STORM	119.38	—	1500	—
SANITARY	118.40	—	525	—
INTERCEPTOR	—	—	100	9.0

METHOD: 2

SCALE: 1:500

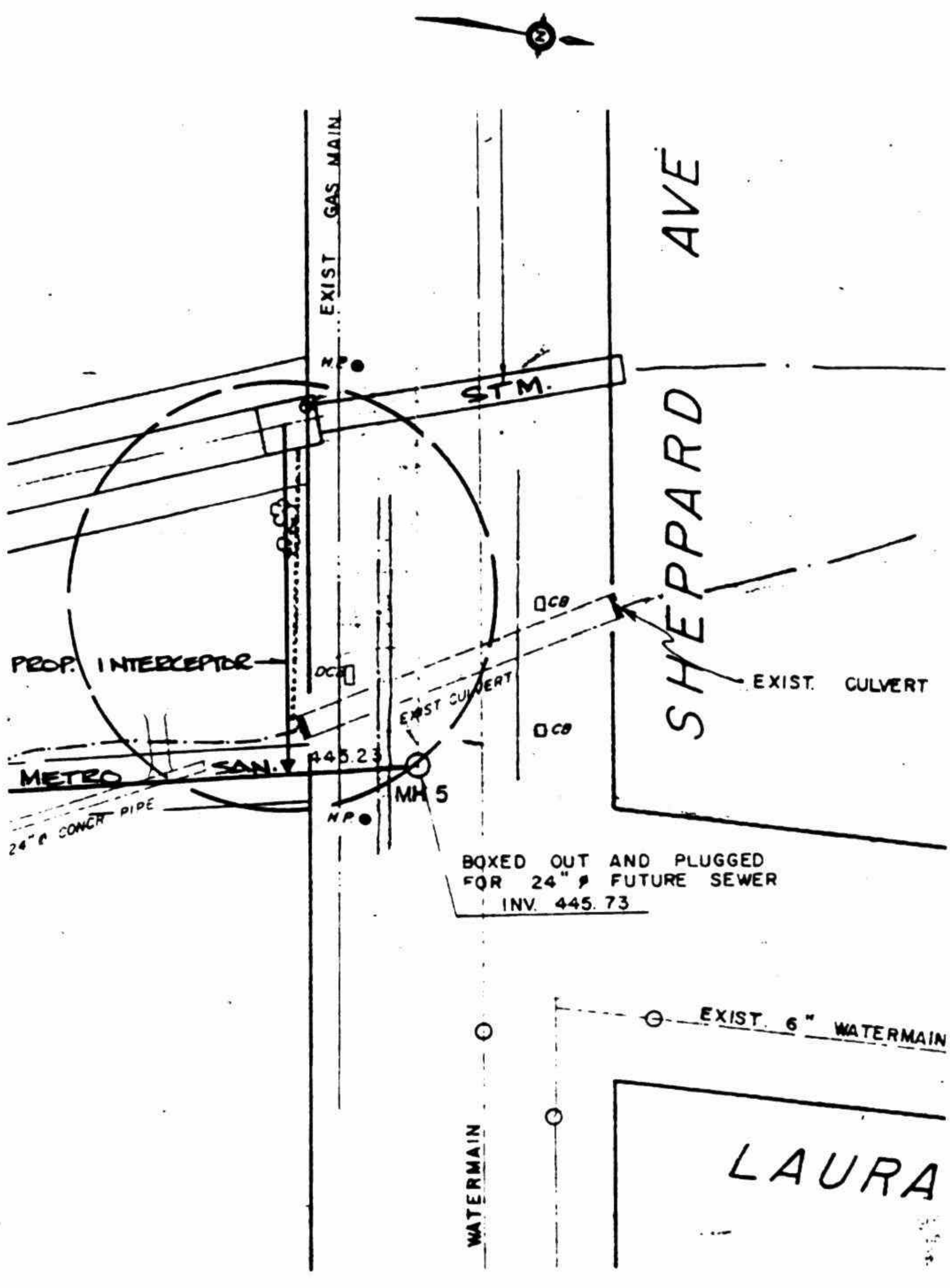
OUTFALL No. N-247

FIGURE No: A-16



	INVERT (m.)	GRADE (%)	SIZE (mm.)	LENGTH (m.)	METHOD: 1	OUTFALL No.
STORM	122·78	—	750	—		0-279
SANITARY	121·32	—	250	—	SCALE: 1:500	
INTERCEPTOR	—	—	100	2·0		

FIGURE No: A-17



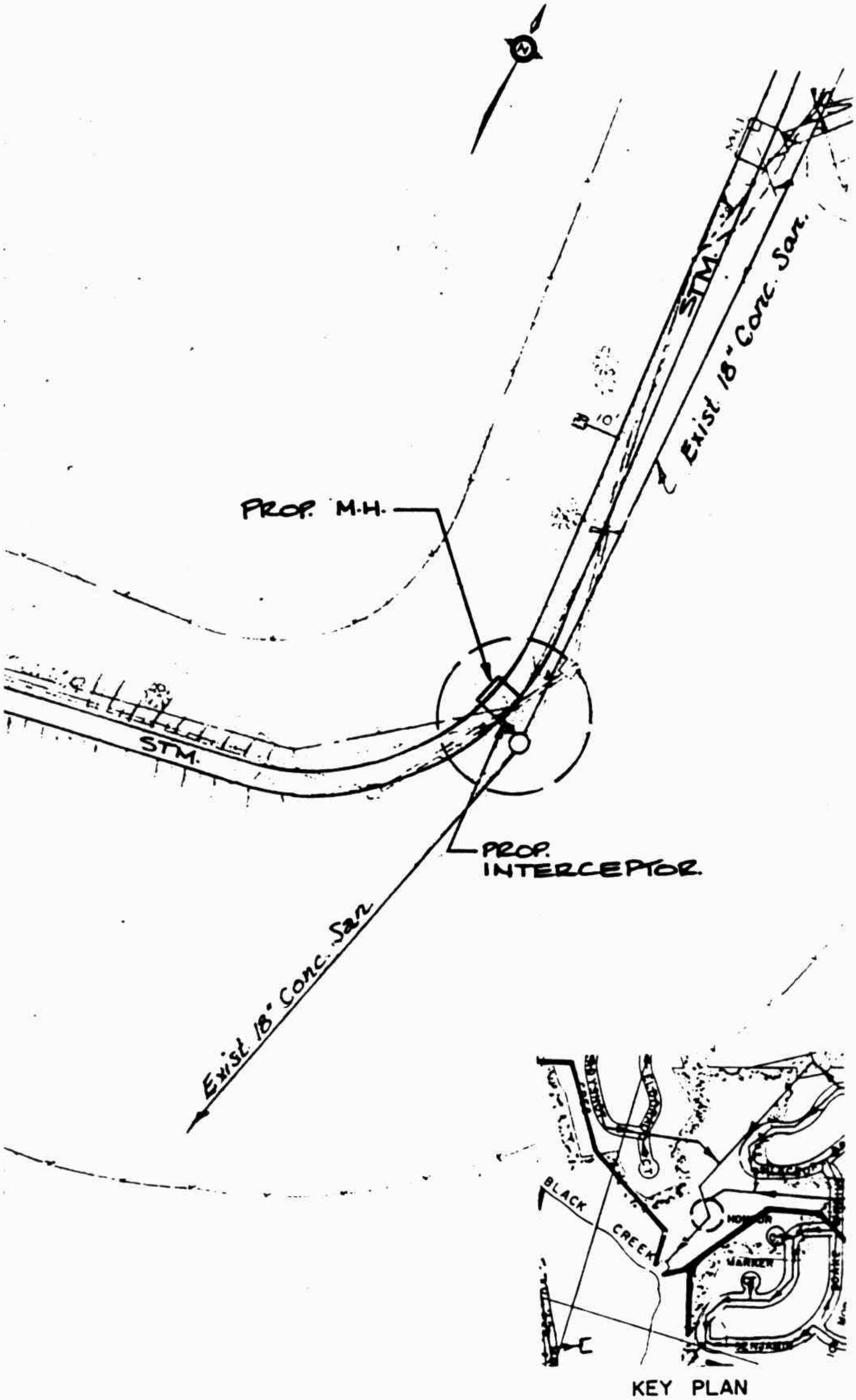
	INVERT (m.)	GRADE (%)	SIZE (mm.)	LENGTH (m.)
STORM	136.30	0.35	2128×1549	—
SANITARY	135.35	0.70	600	—
INTERCEPTOR	—	—	150	24.32

METHOD: 1

SCALE: 1:500

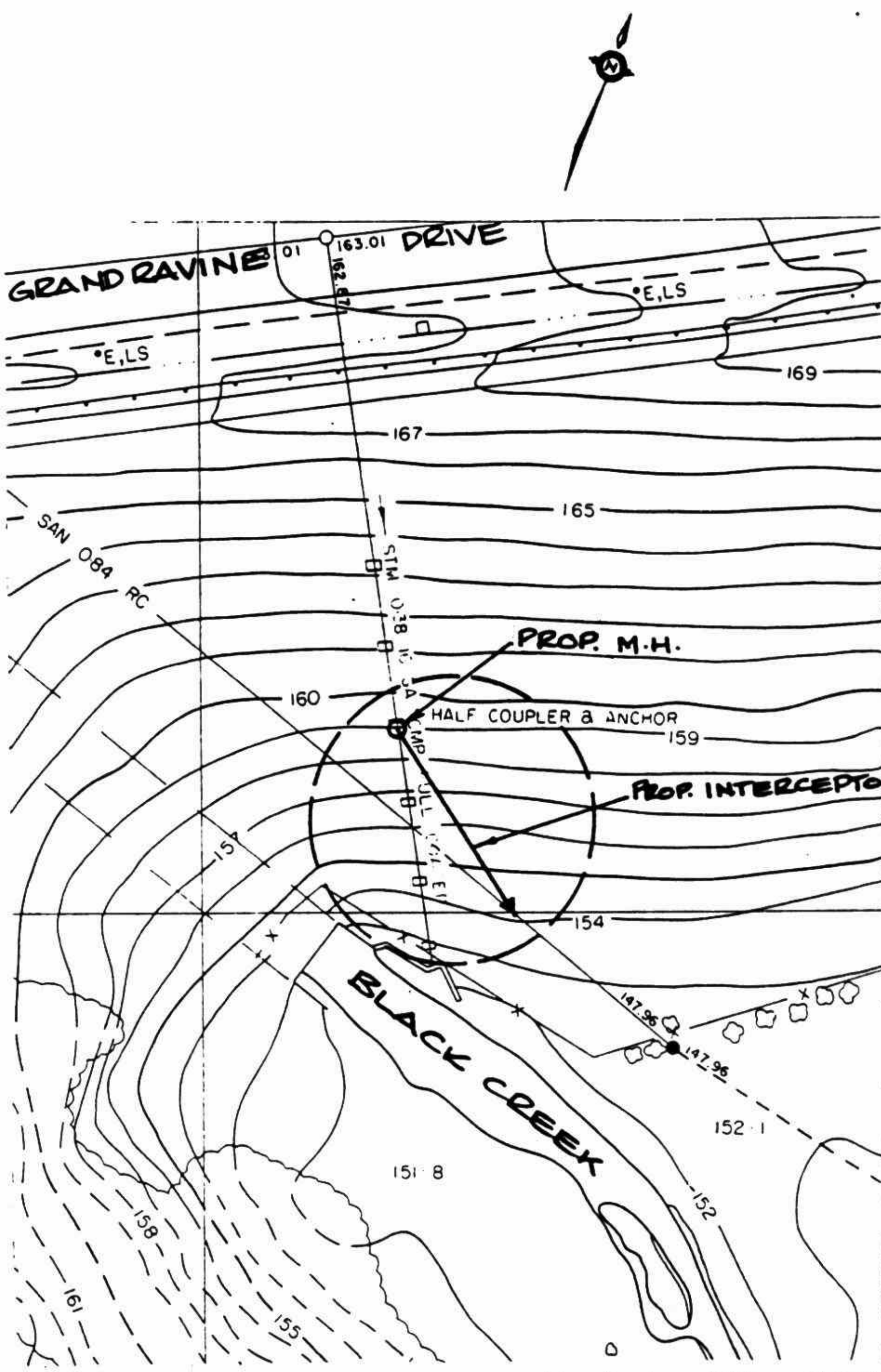
OUTFALL No. 0-341

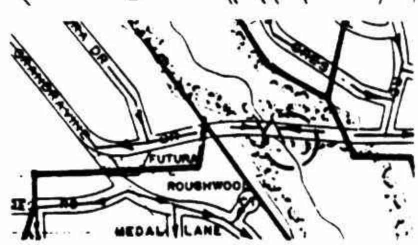
FIGURE No: A-18



	INVERT (m.)	GRADE (%)	SIZE (mm.)	LENGTH (m.)	METHOD: 2	OUTFALL No.
STORM	150·51	1·0	1650	—		P-383
SANITARY	149·57	—	450	—	SCALE: 1:500	
INTERCEPTOR	—	—	200	1·824		

FIGURE No: A-19

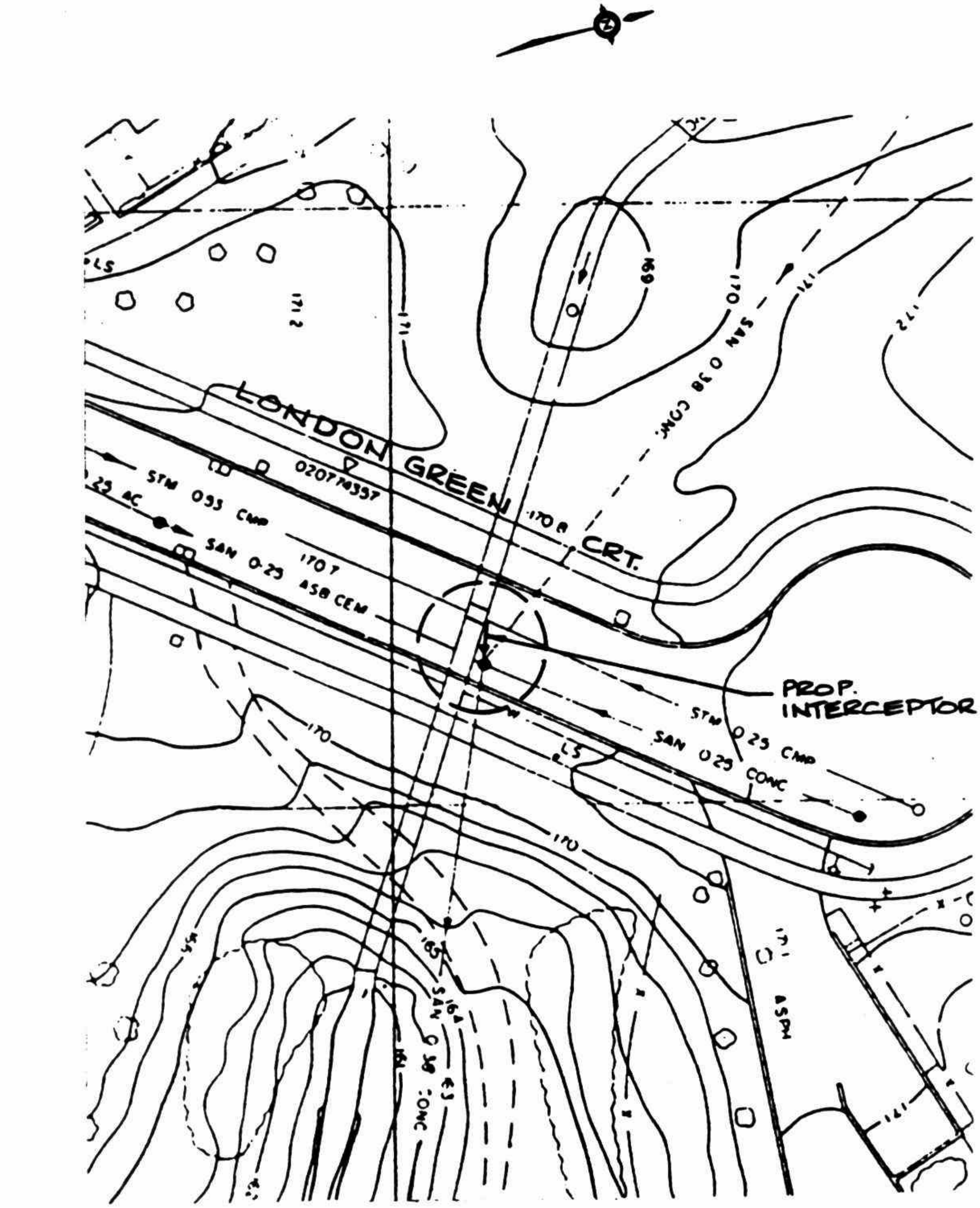




KEY PLAN

	INVERT (m.)	GRADE (%)	SIZE (mm.)	LENGTH (m.)	METHOD: 1	OUTFALL No.
STORM	157.00	—	380	—		P-387
SANITARY	149.00	—	840	—	SCALE: 1:500	
INTERCEPTOR	—	—	100	15		

FIGURE No: A-20



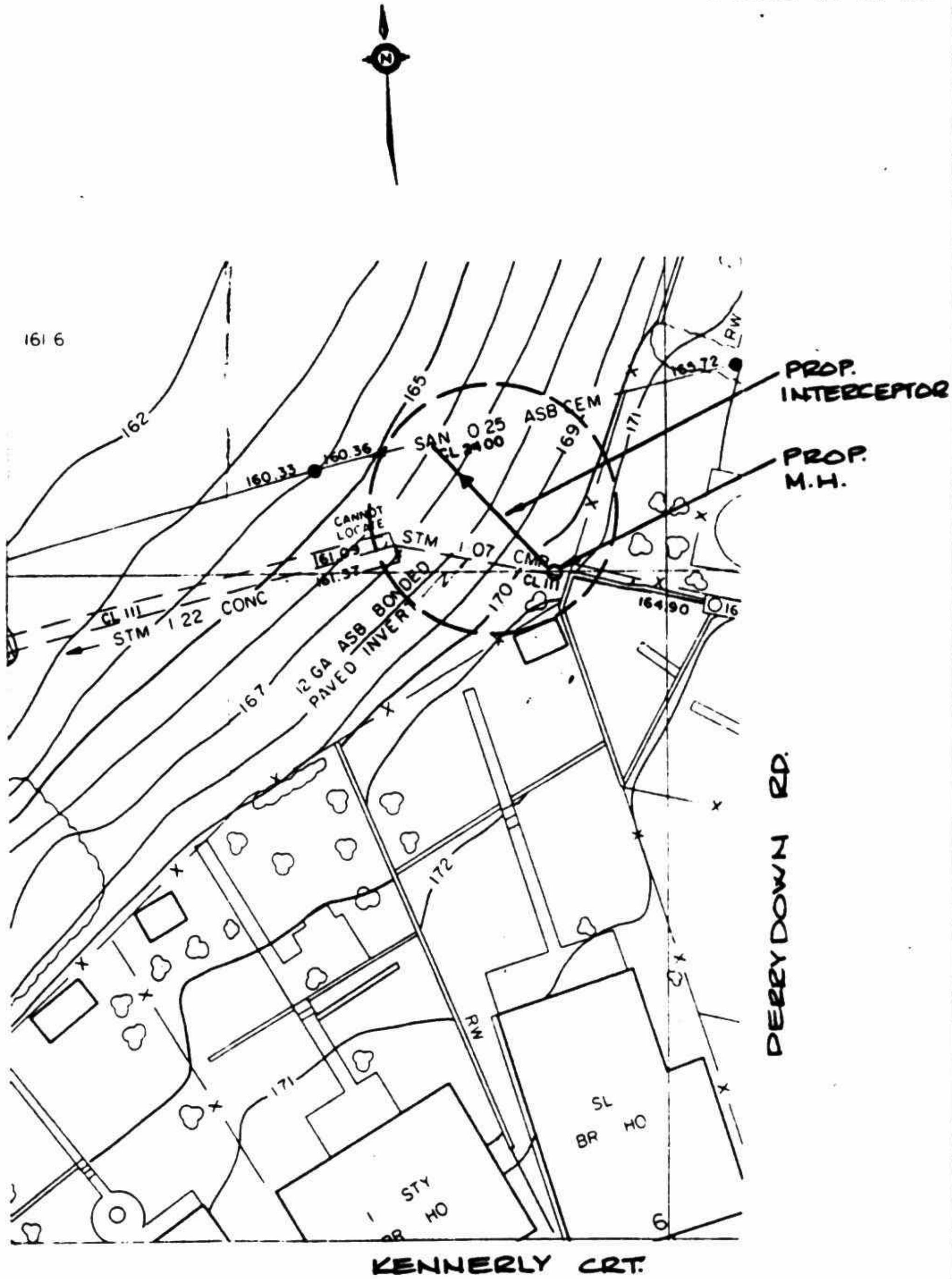
	INVERT (m.)	GRADE (%)	SIZE (mm.)	LENGTH (m.)
STORM	162.79	4.5	1800	—
SANITARY	158.17	—	375	—
INTERCEPTOR	—	—	200	2.0

METHOD: 2

SCALE: 1:500

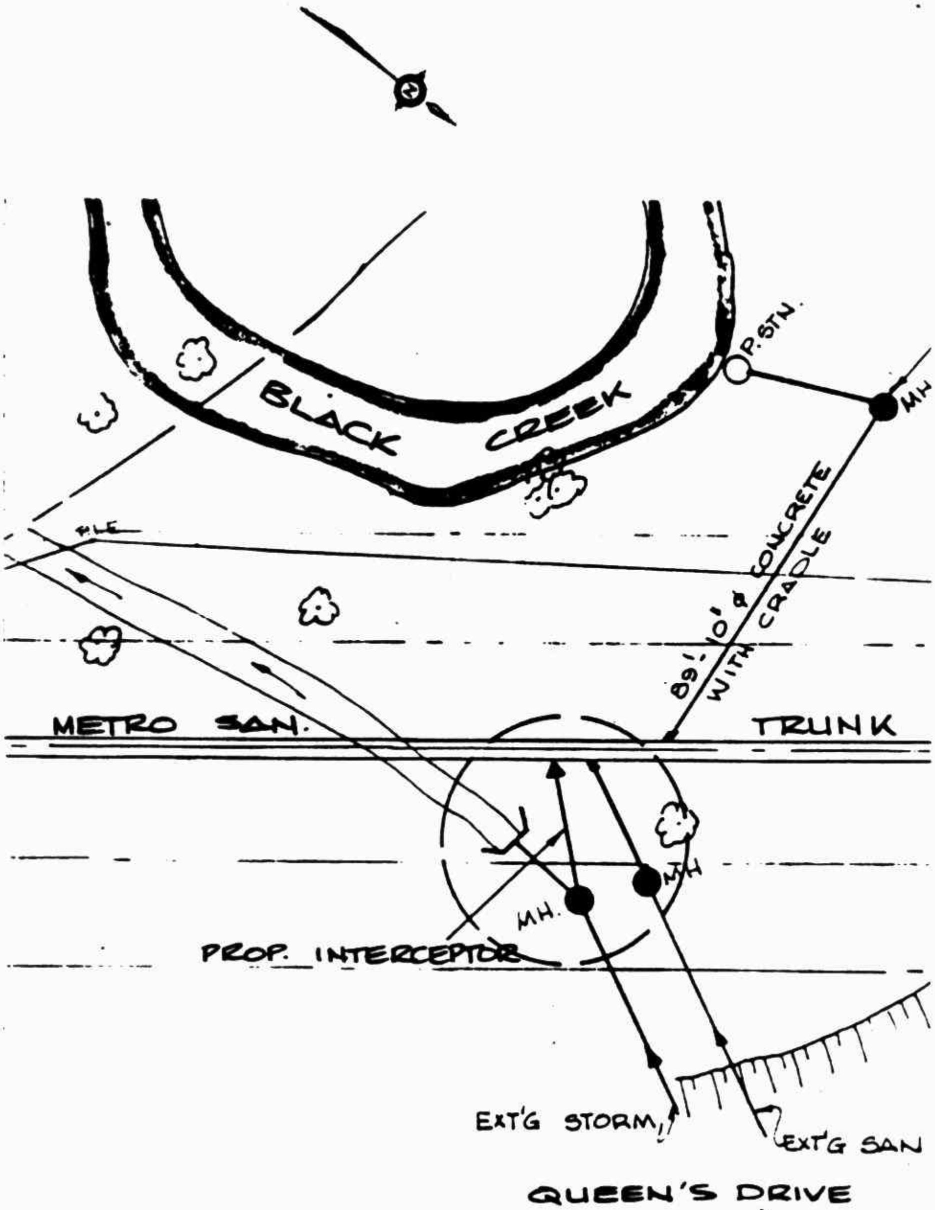
OUTFALL No. P-393

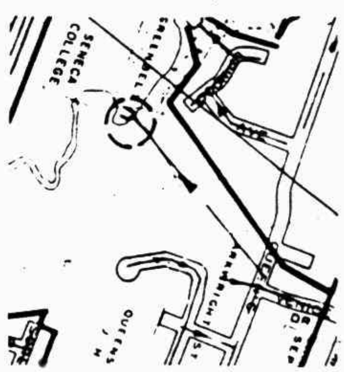
FIGURE No: A-21



	INVERT (m.)	GRADE (%)	SIZE (mm.)	LENGTH (m.)	METHOD: 1	OUTFALL No.
STORM	163.4	14.7	1070	—		P395
SANITARY	161.0	16.5	250	—	SCALE: 1:500	
INTERCEPTOR	—	—	100	15		

FIGURE No: A-22

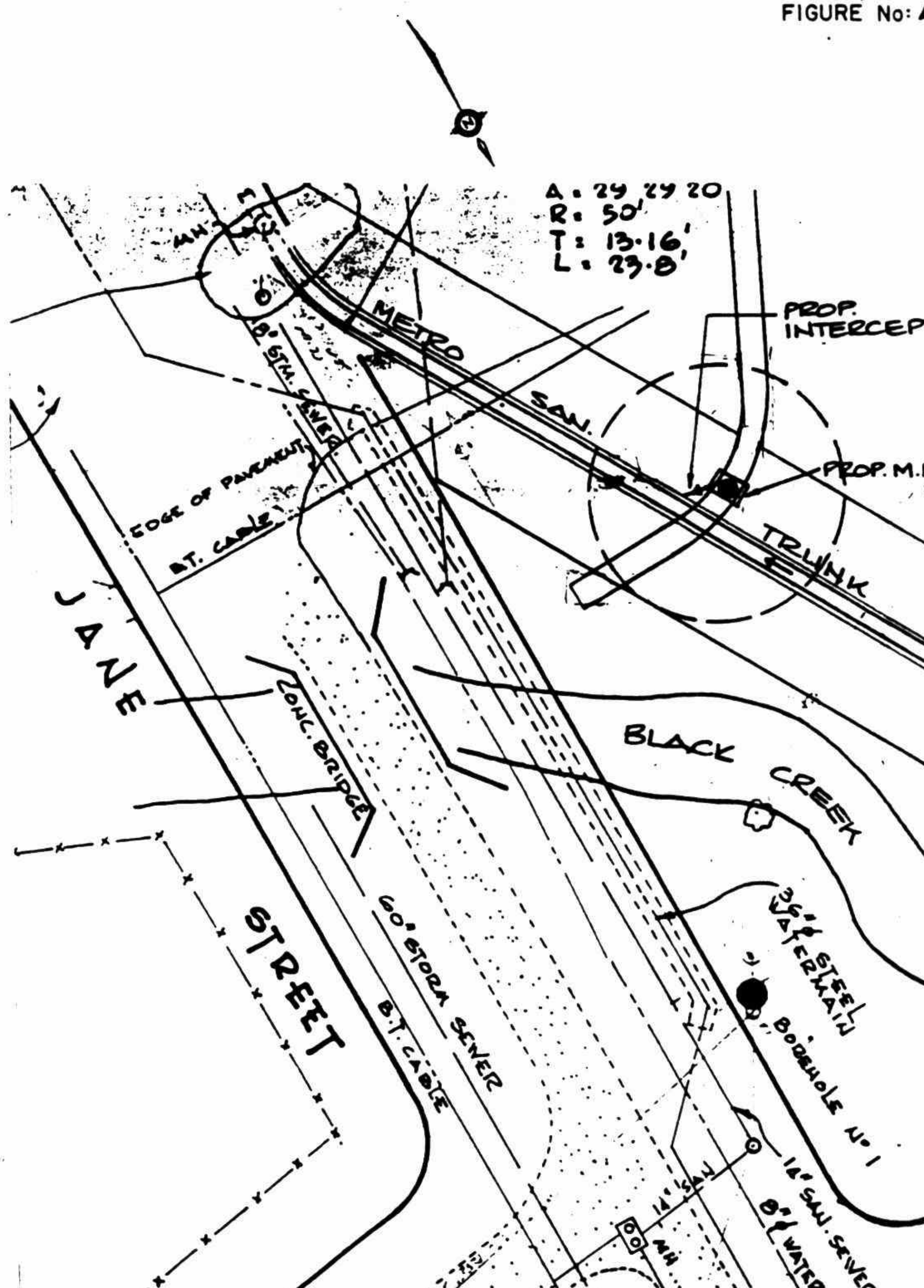




KEY PLAN

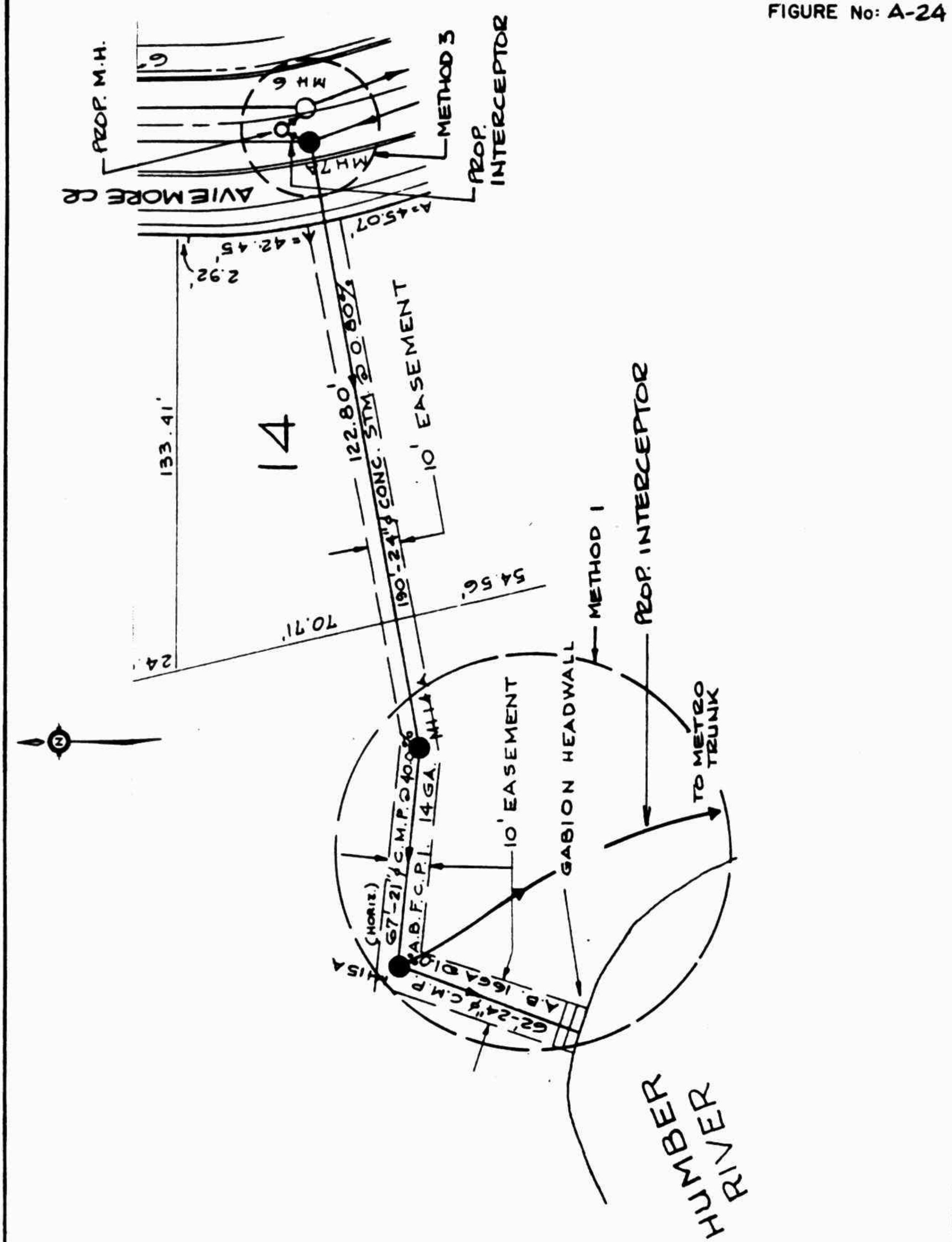
	INVERT (m.)	GRADE (%)	SIZE (mm.)	LENGTH (m.)	METHOD: 2	OUTFALL No.
STORM	—	—	3120	—		N-225
SANITARY	112.94	0.38	1350	—	SCALE: 1:500	
INTERCEPTOR	—	—	200	9.1		

FIGURE No: A-23



	INVERT (m.)	GRADE (%)	SIZE (mm.)	LENGTH (m.)	METHOD: 2	OUTFALL No.
STORM	122.46	0.11	1800	—		0-301
SANITARY	120.23	0.40	1050	—	SCALE: 1:500	
INTERCEPTOR	—	—	200	3.0		

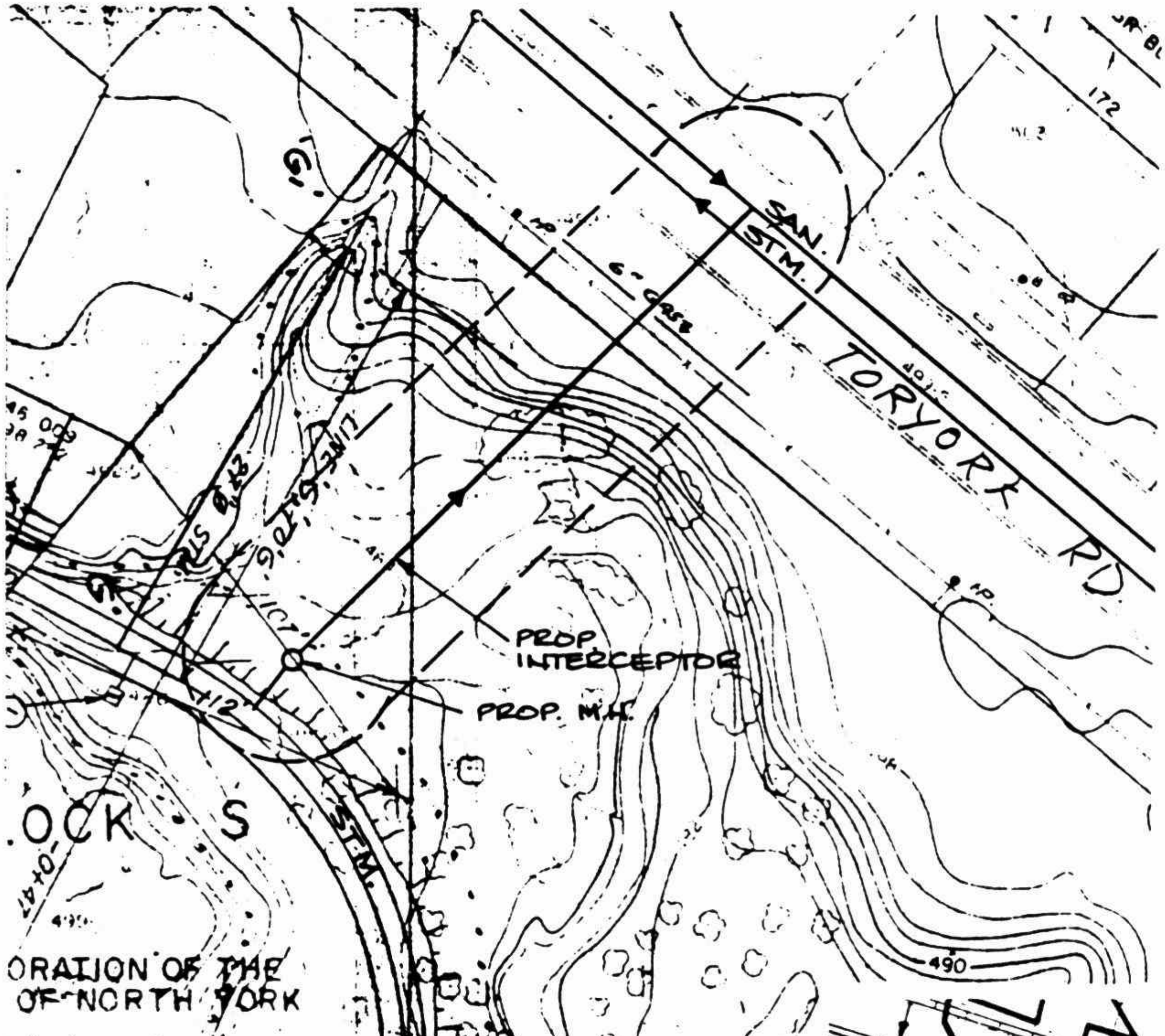
FIGURE No: A-24



	INVERT (m.)	GRADE (%)	SIZE (mm.)	LENGTH (m.)	METHOD: 1	OUTFALL No.
STORM	130·57	1·0	600	—		H-441
SANITARY	124·32	1·07	1050	—	SCALE: 1:500	
INTERCEPTOR	—	—	100	182·4		

	INVERT (m.)	GRADE (%)	SIZE (mm.)	LENGTH (m.)	METHOD: 3	OUTFALL No.
STORM	139·51	0·8	600	—		H-441
SANITARY	141·88	0.25	250	—	SCALE: 1:500	
INTERCEPTOR	—	—	100	6·0		

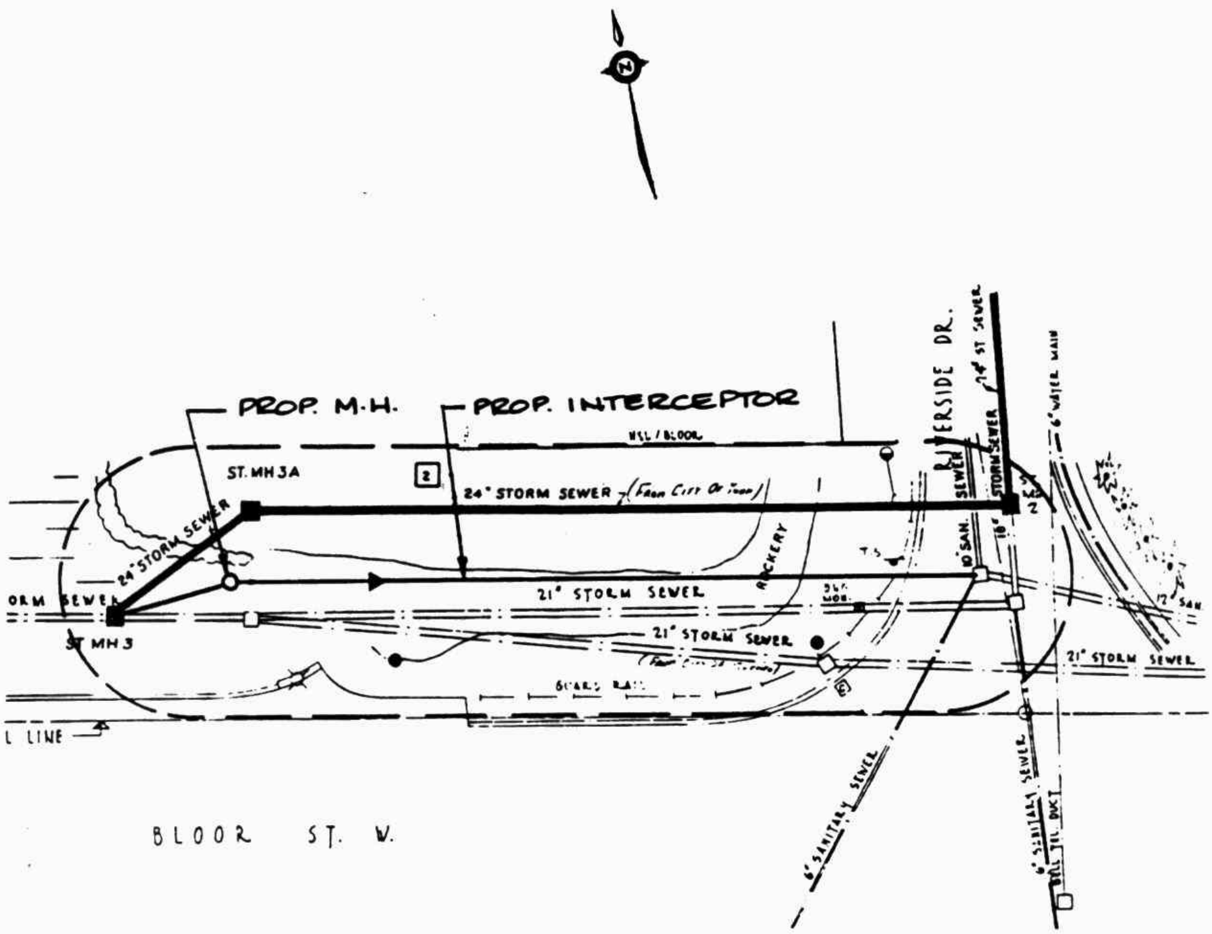
FIGURE No: A-25



KEY PLAN

	INVERT (m.)	GRADE (%)	SIZE (mm.)	LENGTH (m.)	METHOD: 3	OUTFALL No.
STORM	143.64	0.50	3000	—		
SANITARY	—	—	—	—	SCALE: 1:500	G-502
INTERCEPTOR	—	—	150	54.7		

FIGURE No: A-26



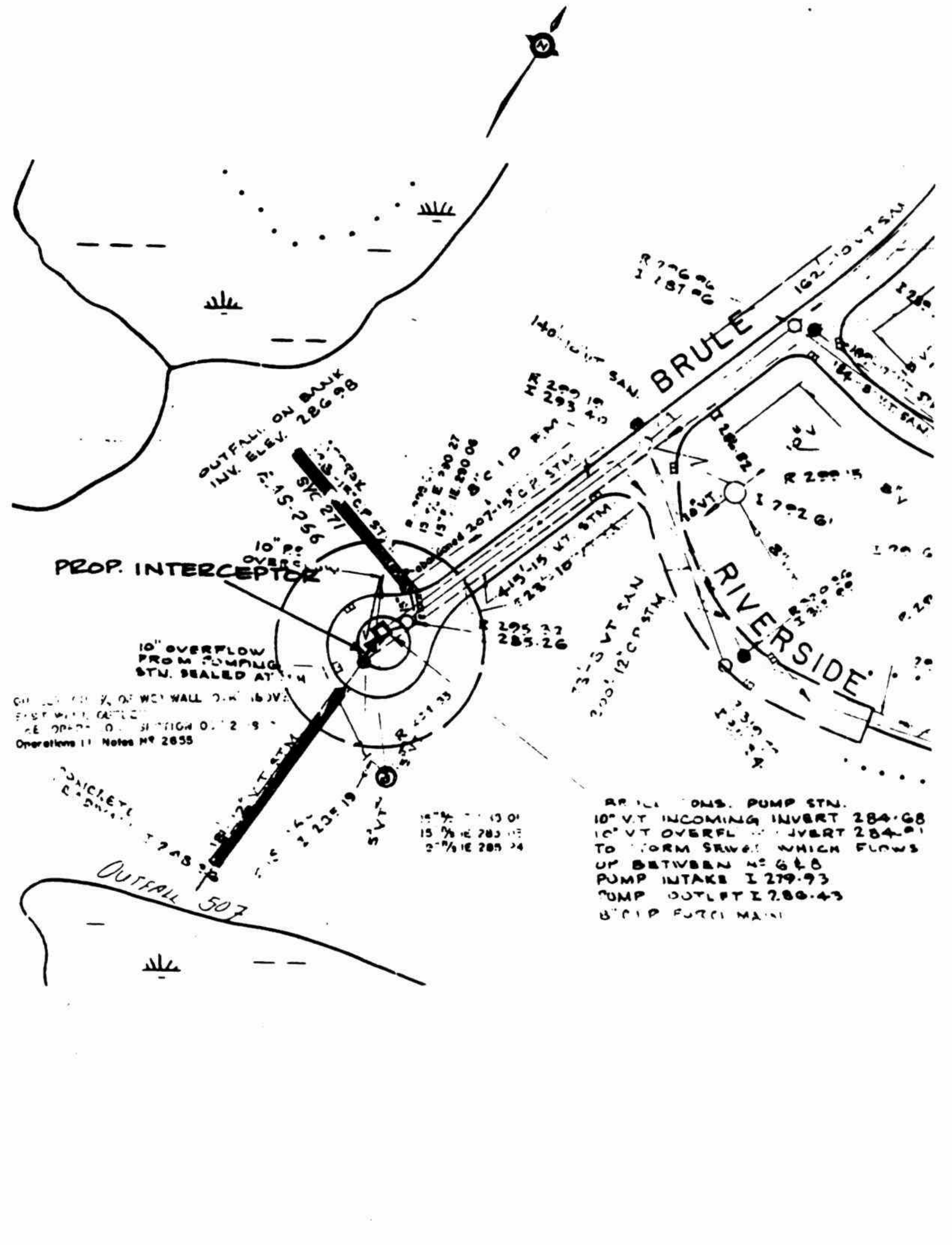
	INVERT (m.)	GRADE (%)	SIZE (mm.)	LENGTH (m.)
STORM	79.78	29.2	450	—
SANITARY	85.85	—	300	—
INTERCEPTOR	—	—	150	60.8

METHOD: 3

SCALE: 1:500

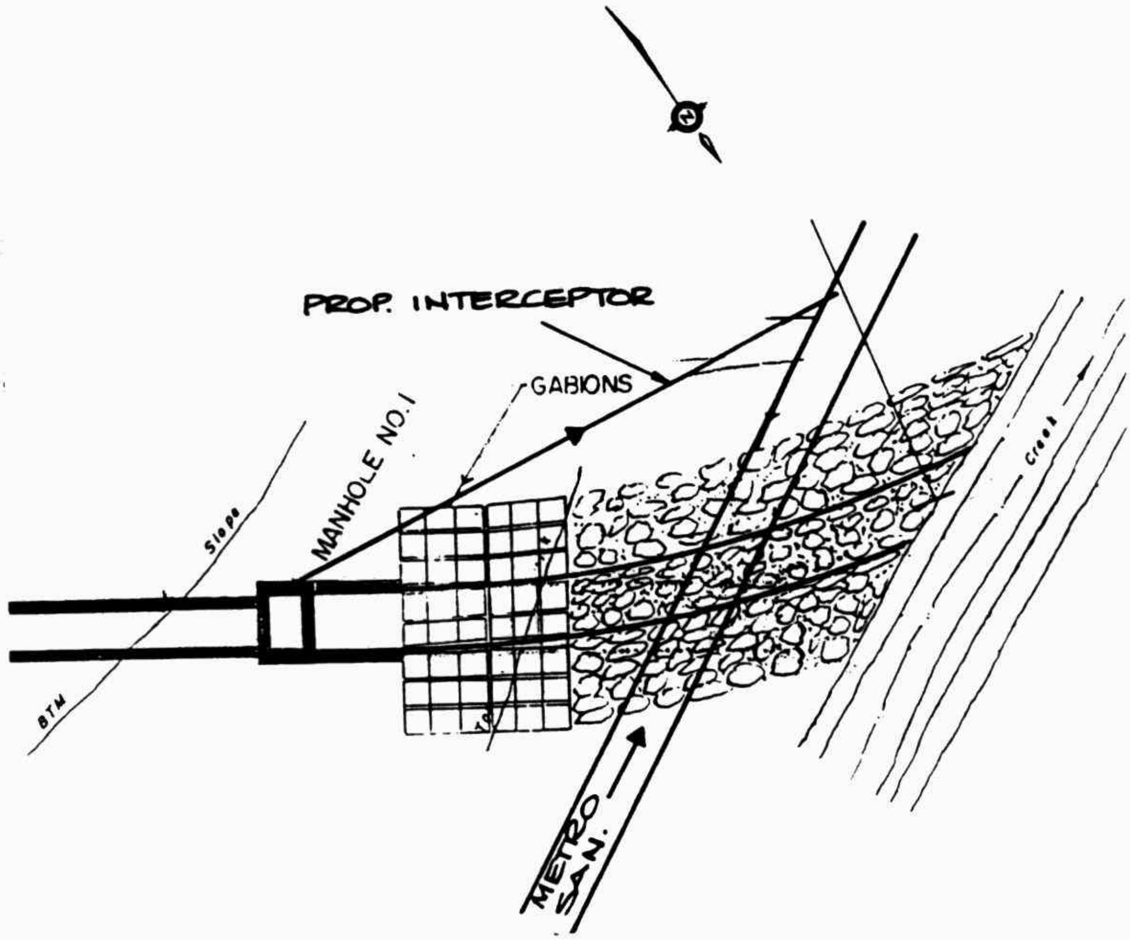
OUTFALL No. B-509

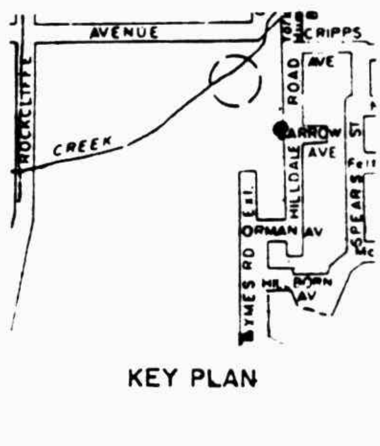
FIGURE No: A-27



	INVERT (m.)	GRADE (%)	SIZE (mm.)	LENGTH (m.)	METHOD: 3	OUTFALL No.
STORM	86.65	—	375	—		A-507
SANITARY	86.72	—	250	—	SCALE: 1:1250	
INTERCEPTOR	—	—	150	7.6		

FIGURE No: A-28

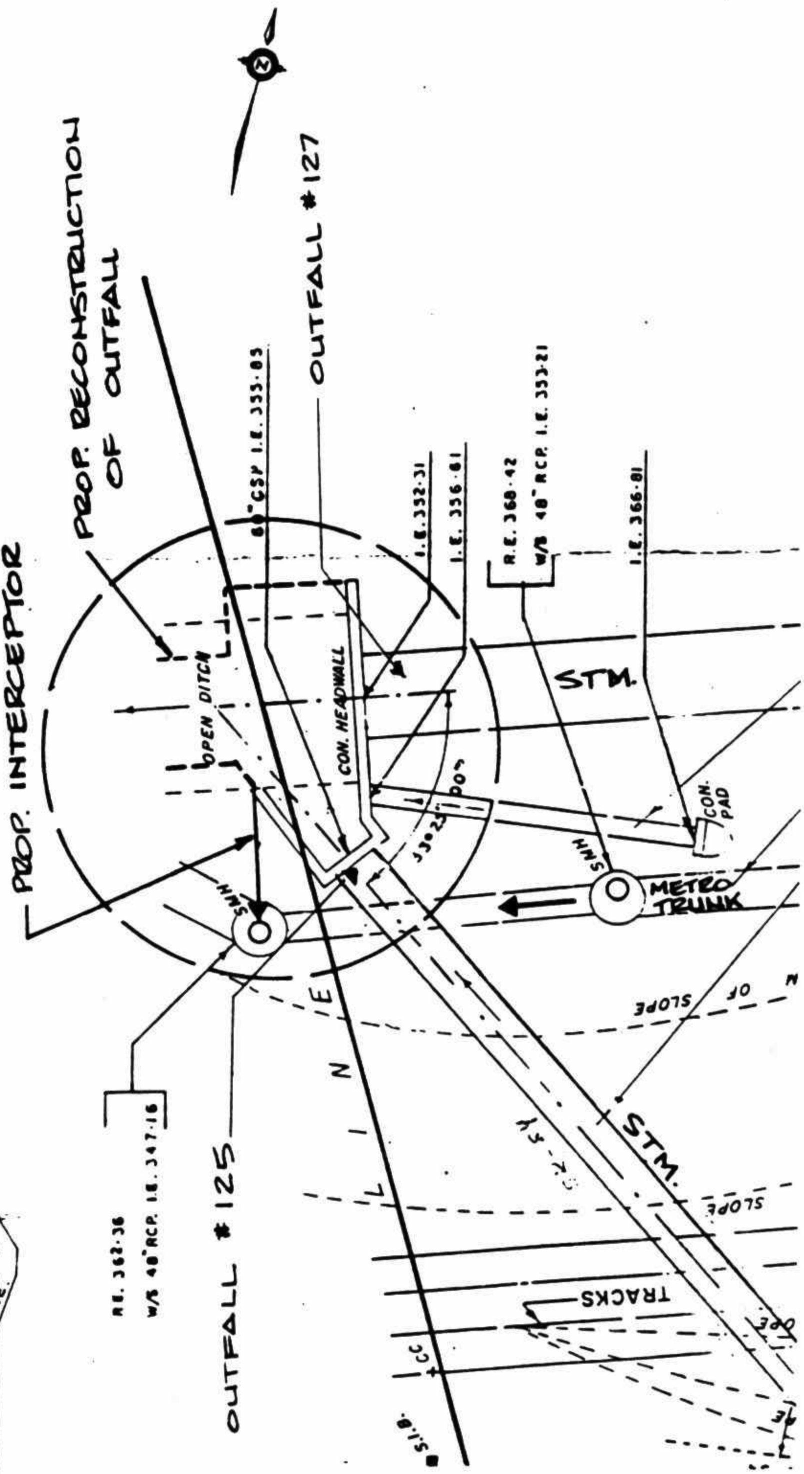


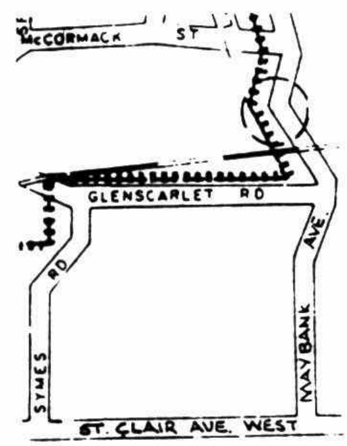


KEY PLAN

	INVERT (m.)	GRADE (%)	SIZE (mm.)	LENGTH (m.)	METHOD: 1	OUTFALL No.
STORM	105.68	3.94	1350	—		E-123
SANITARY	101.87	—	1200	—	SCALE: 1:250	
INTERCEPTOR	—	—	100	21.3		

FIGURE No: A-29



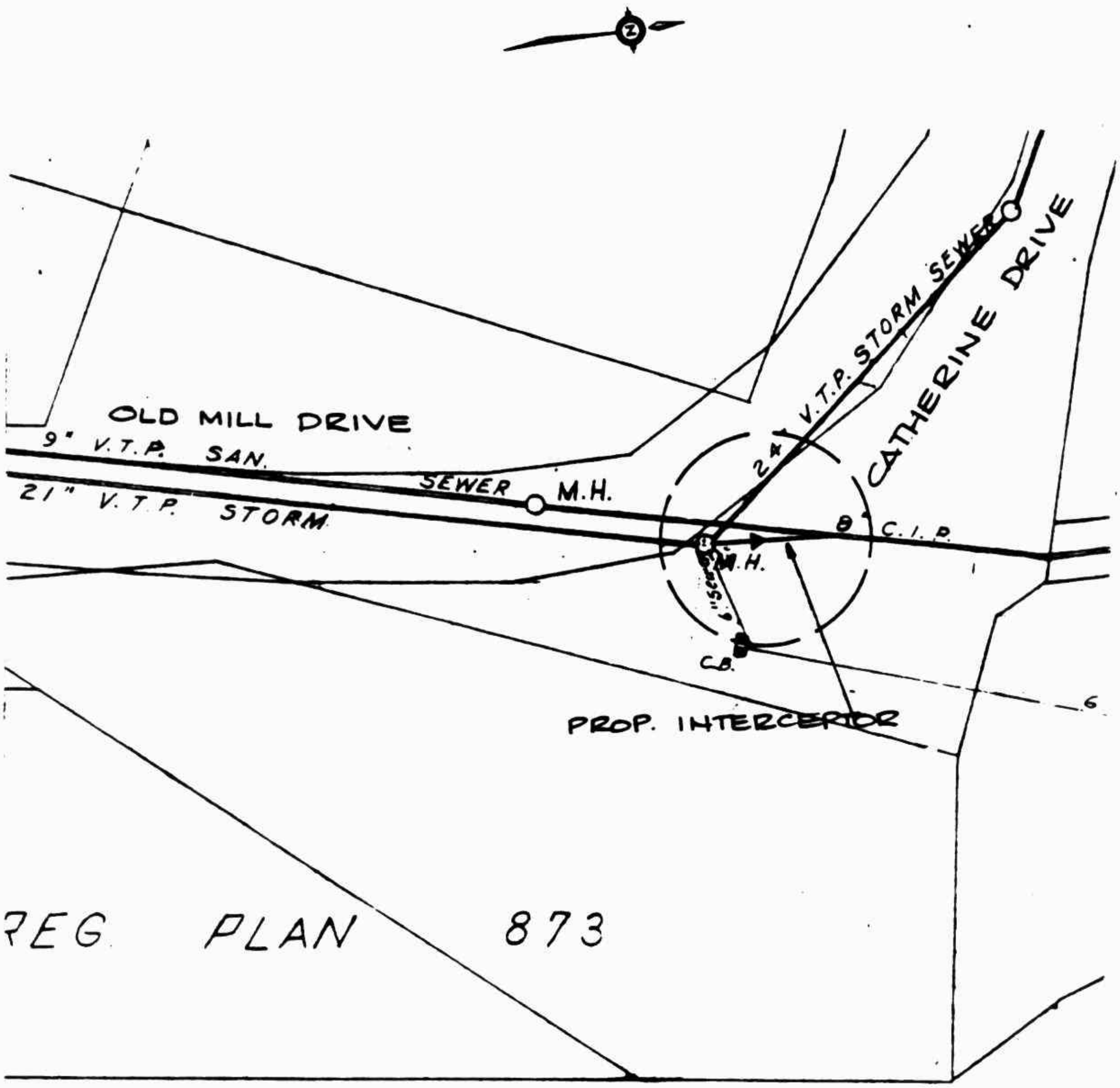


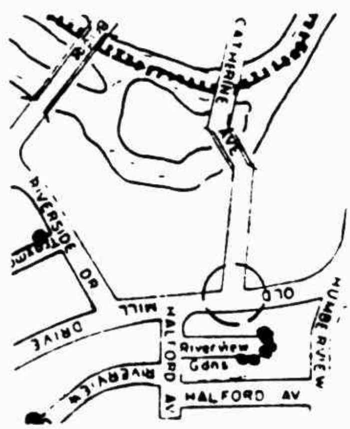
KEY PLAN

	INVERT (m.)	GRADE (%)	SIZE (mm.)	LENGTH (m.)	METHOD: 2	OUTFALL No.
STORM	108.18	—	1500	—		L-125
SANITARY	105.54	—	1200	—	SCALE: 1:250	
INTERCEPTOR	—	—	200	3.8		

	INVERT (m.)	GRADE (%)	SIZE (mm.)	LENGTH (m.)	METHOD: 2	OUTFALL No.
STORM	107.10	—	2100	—		L-127
SANITARY	105.54	—	1200	—	SCALE: 1:250	
INTERCEPTOR	—	—	200	3.8		

FIGURE No: A-30





KEY PLAN

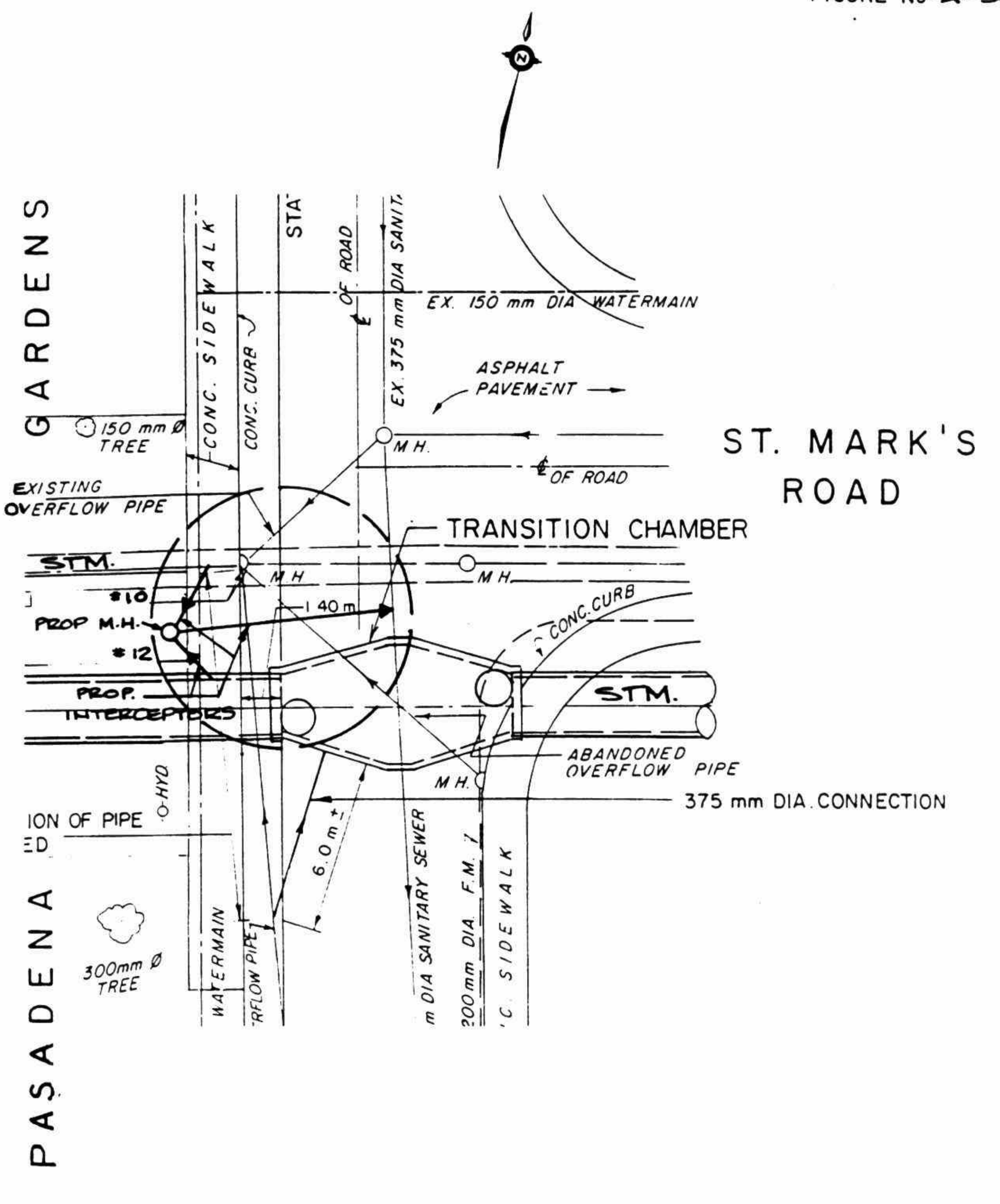
	INVERT (m.)	GRADE (%)	SIZE (mm.)	LENGTH (m.)
STORM	81.30	4.27	600	—
SANITARY	79.00	14.32	200	—
INTERCEPTOR	—	—	100	12.16

METHOD: 1

SCALE: 1:500

OUTFALL No. B-6

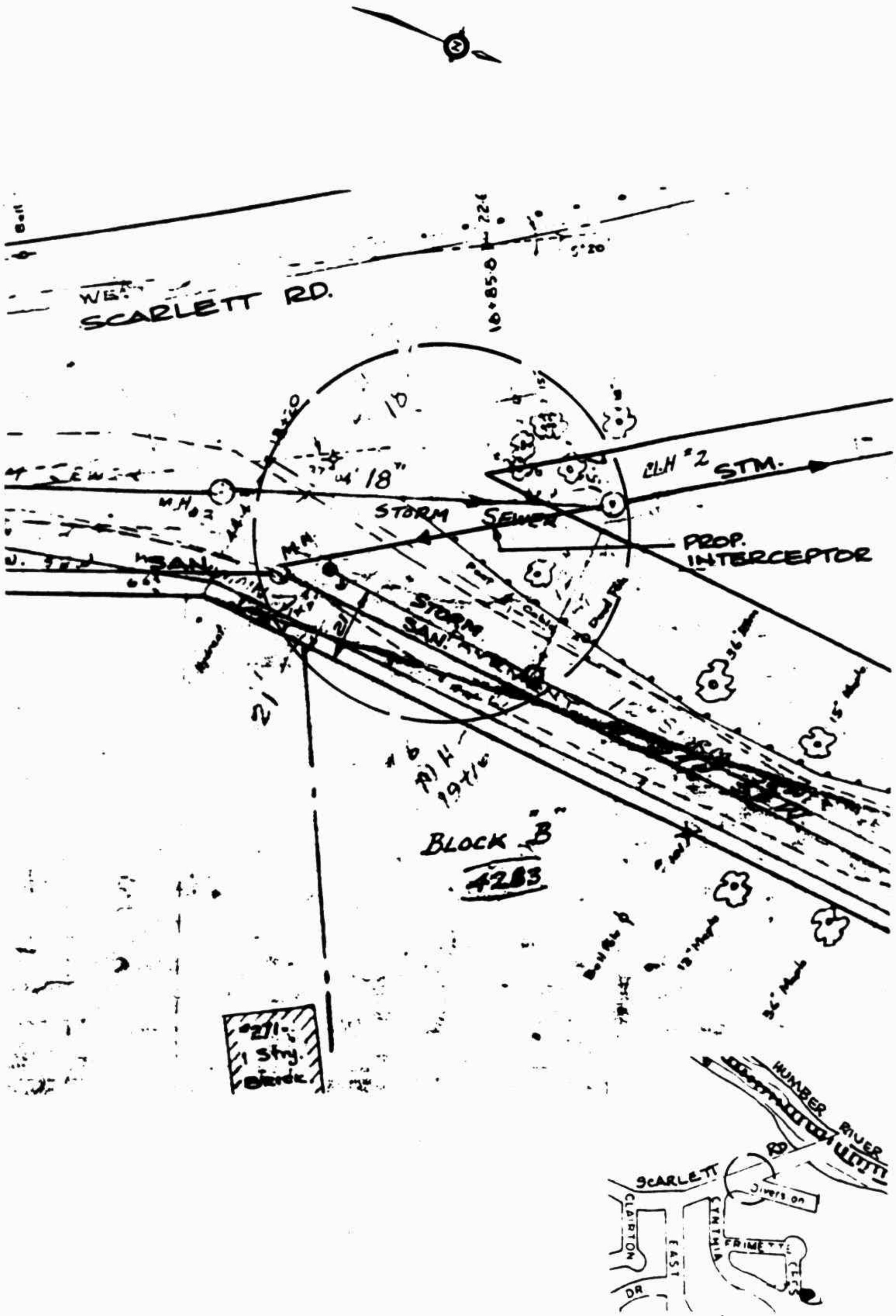
FIGURE No: A-31



	INVERT (m.)	GRADE (%)	SIZE (mm.)	LENGTH (m.)	METHOD: 3	OUTFALL No.
STORM	84.96	0.65	1950	—		B-10
SANITARY	—	—	450	—	SCALE: 1:200	
INTERCEPTOR	—	—	100	9		

	INVERT (m.)	GRADE (%)	SIZE (mm.)	LENGTH (m.)	METHOD: 3	OUTFALL No.
STORM	85.60	—	1350	—		B-12
SANITARY	—	—	450	—	SCALE: 1:200	
INTERCEPTOR	—	—	100	9		

FIGURE No: A-32



KEY PLAN

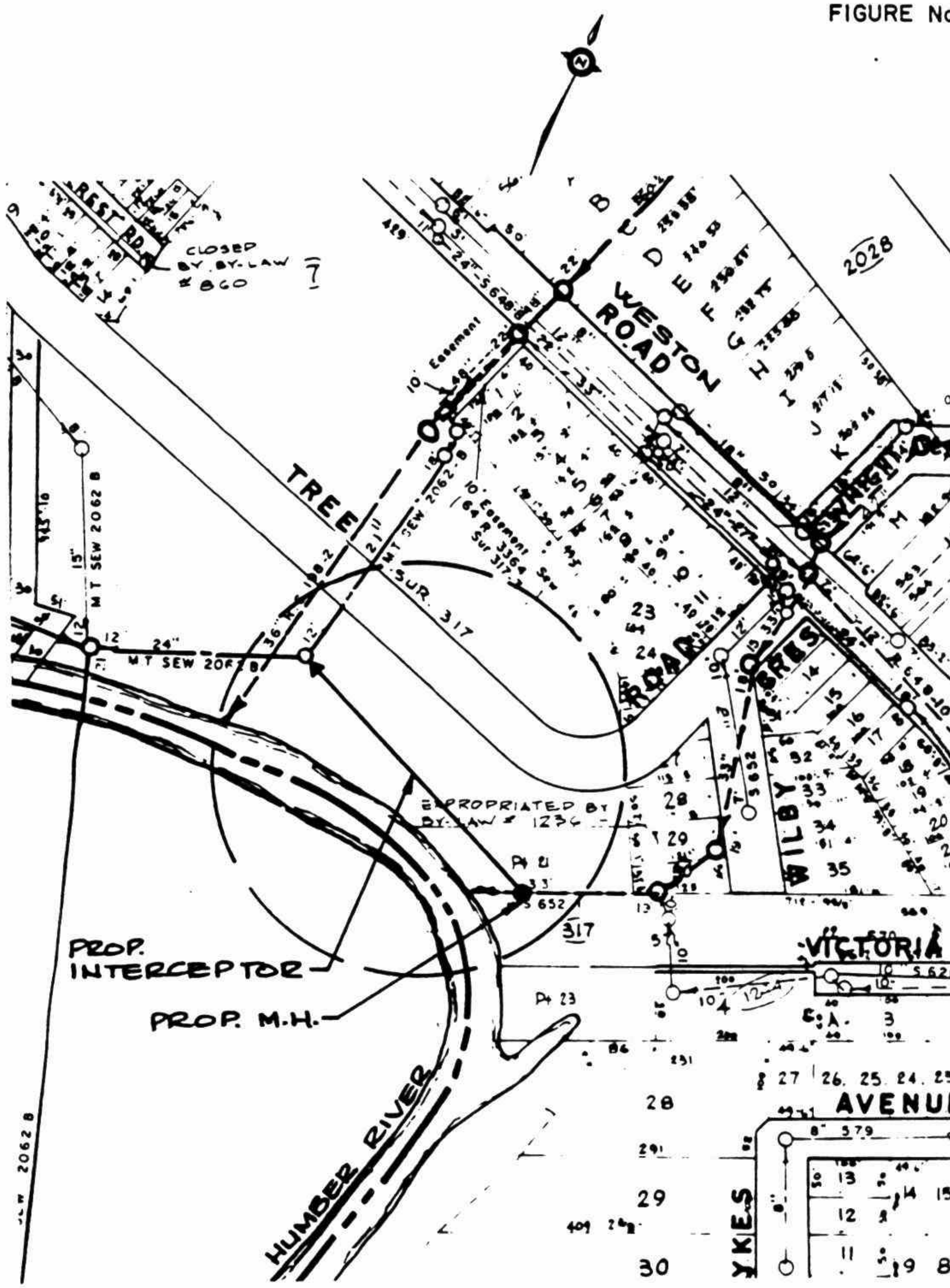
	INVERT (m.)	GRADE (%)	SIZE (mm.)	LENGTH (m.)
STORM	97.80	0.46	450	—
SANITARY	97.19	0.63	230	—
INTERCEPTOR	—	—	100	27.36

METHOD: 2

SCALE: 1:500

OUTFALL No. C-32

FIGURE No: A-33



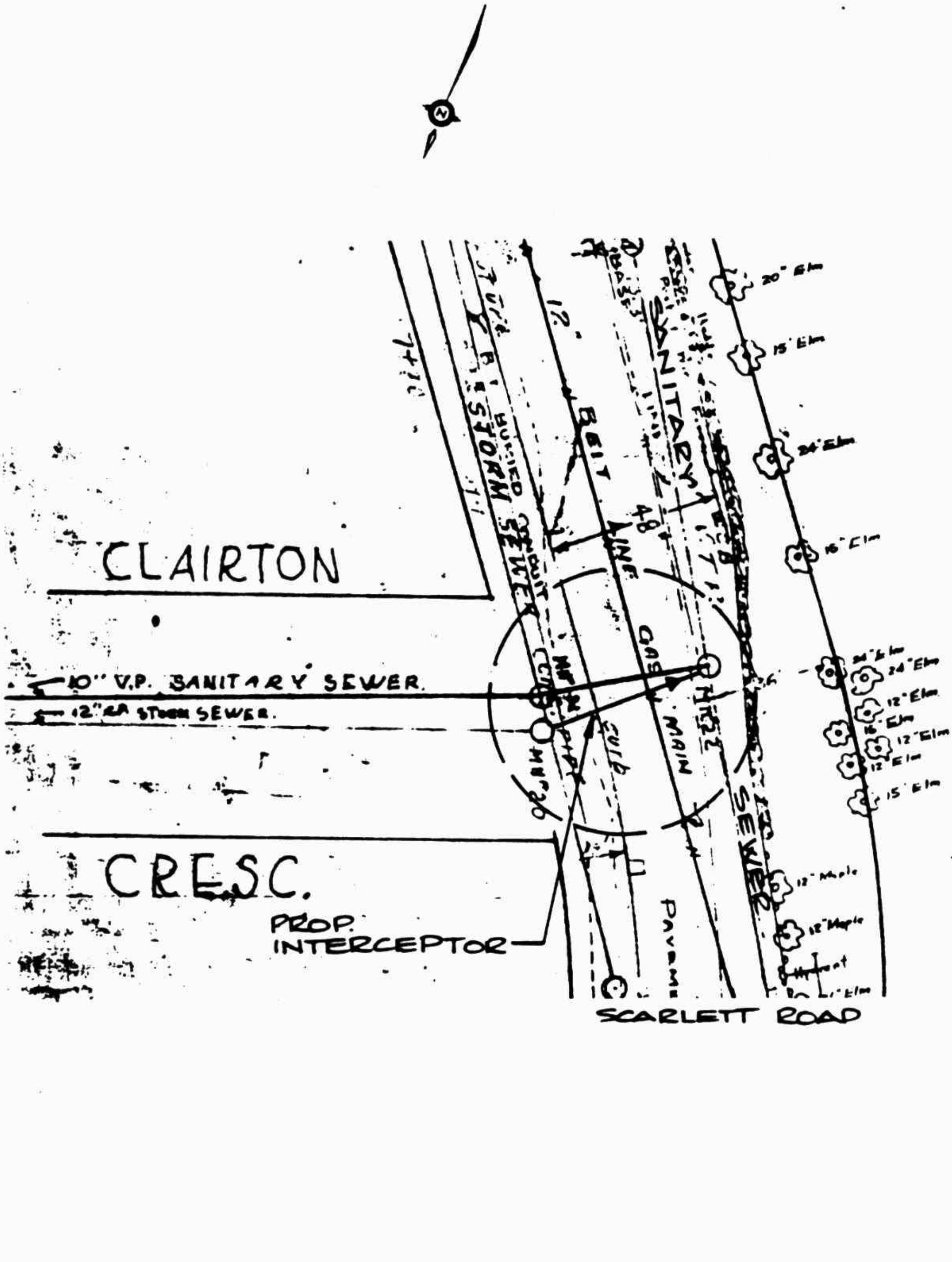
	INVERT (m.)	GRADE (%)	SIZE (mm.)	LENGTH (m.)
STORM	118.86	—	825	—
SANITARY	110.02	1.4	600	—
INTERCEPTOR	—	—	100	121.9

METHOD: 2

SCALE: 1:2500

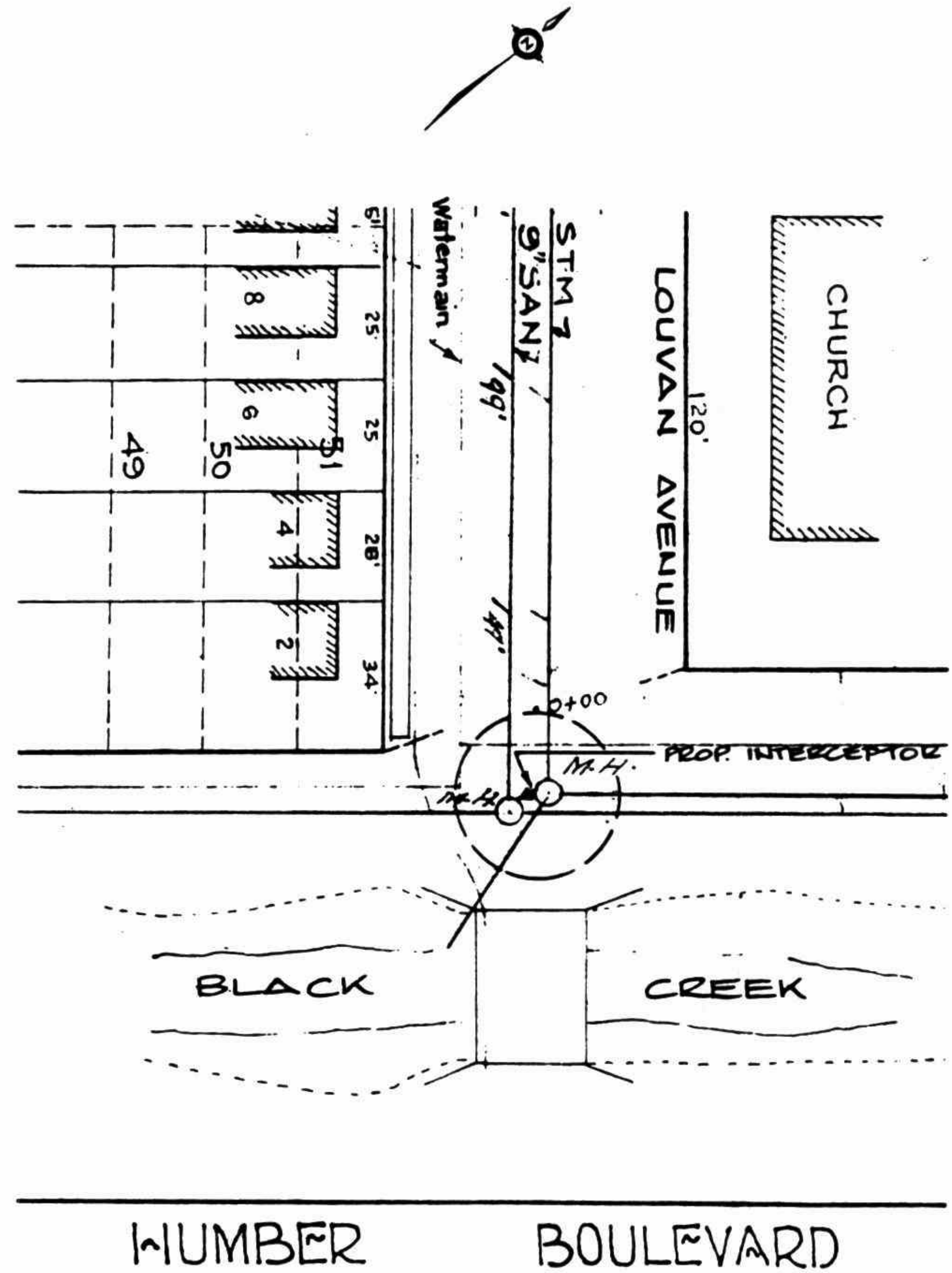
OUTFALL No. D-37

FIGURE No: A-34



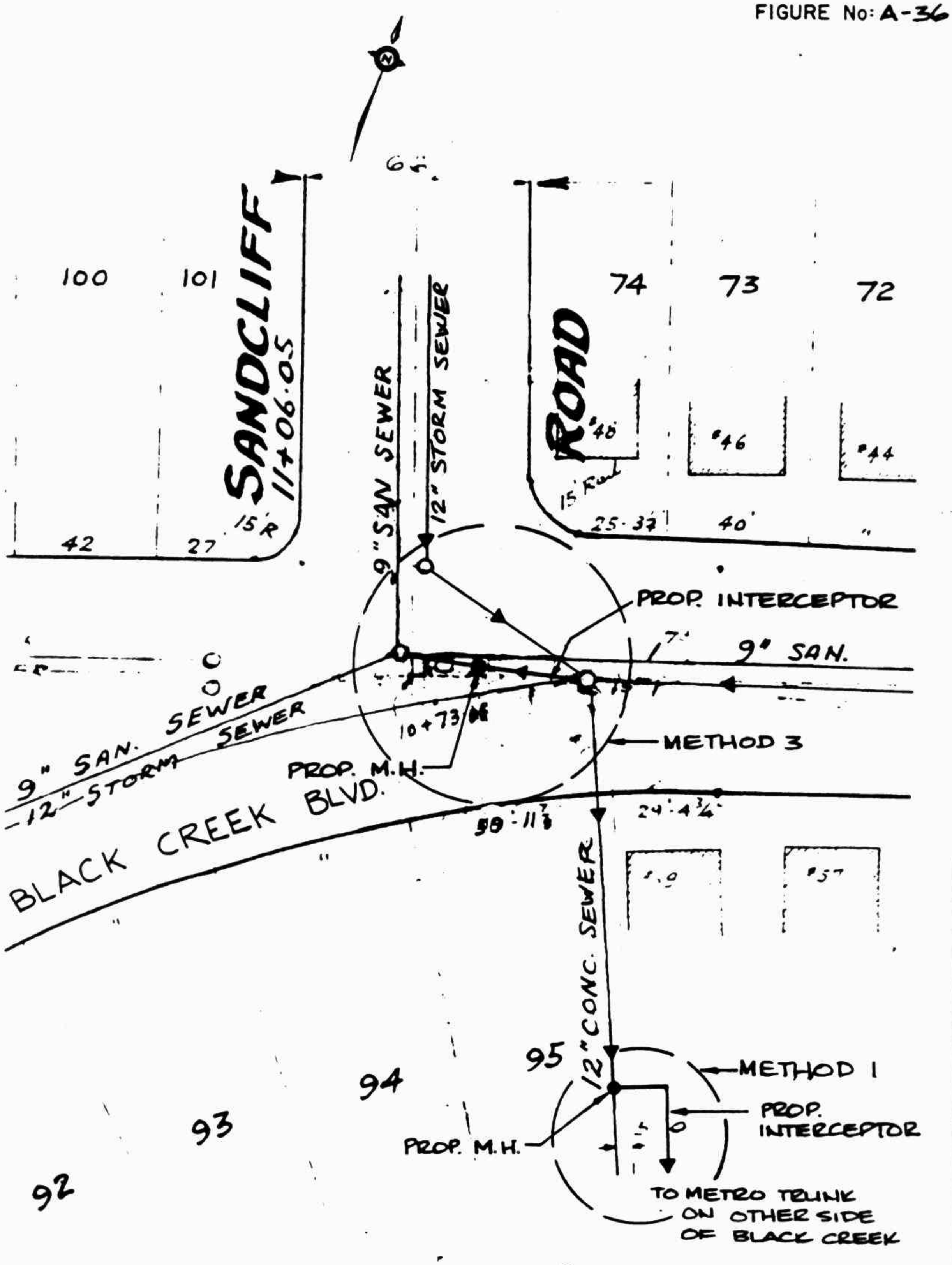
	INVERT (m.)	GRADE (%)	SIZE (mm.)	LENGTH (m.)	METHOD: 2	OUTFALL No.
STORM	96.20	0.47	675	—		L-69
SANITARY	95.12	0.40	230	—	SCALE: 1:500	
INTERCEPTOR	—	—	100	12		

FIGURE No: **A-35**



	INVERT (m.)	GRADE (%)	SIZE (mm.)	LENGTH (m.)	METHOD: 2	OUTFALL No.
STORM	101.42	—	600	—		
SANITARY	100.41	0.53	225	—	SCALE: 1:500	L-135
INTERCEPTOR	—	—	100	1.52		

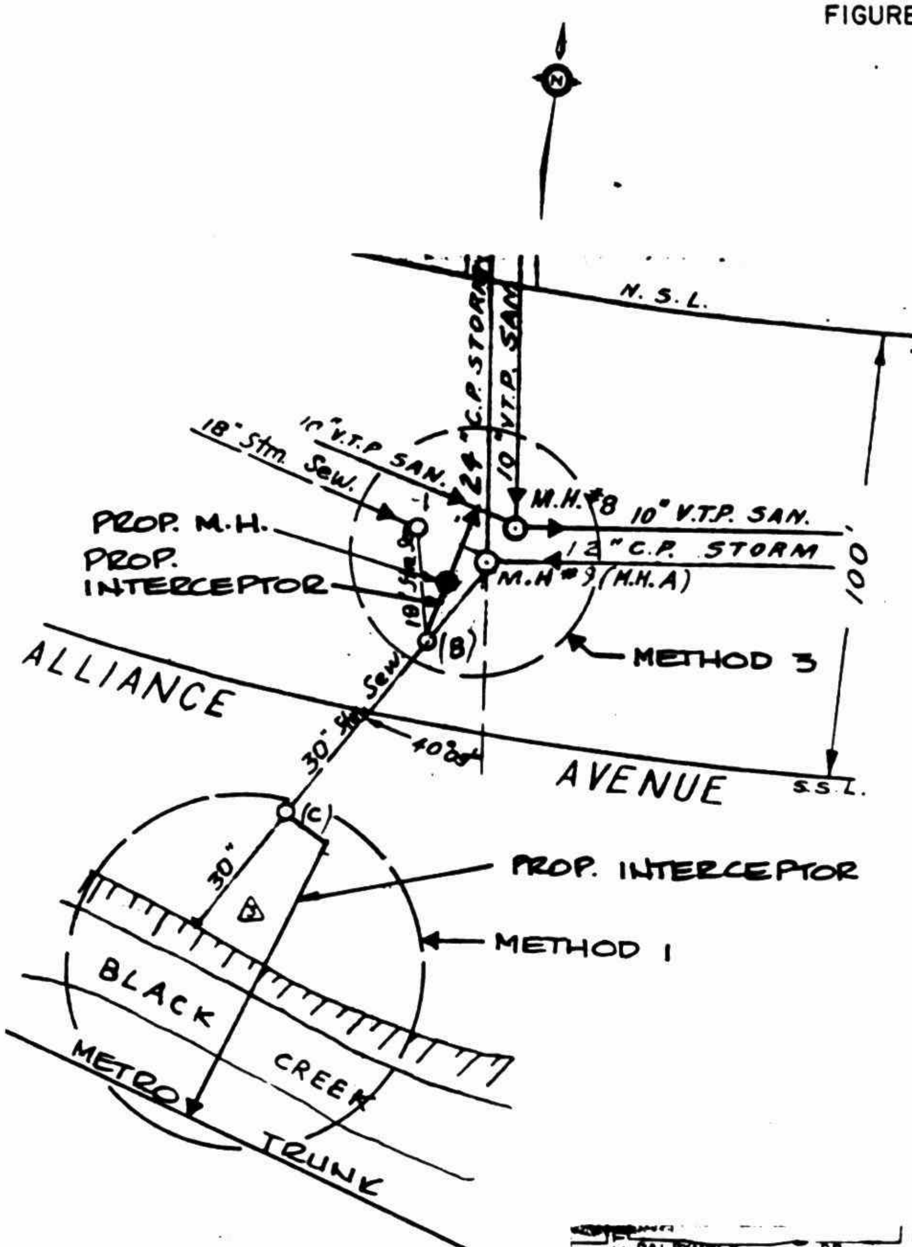
FIGURE No: A-36

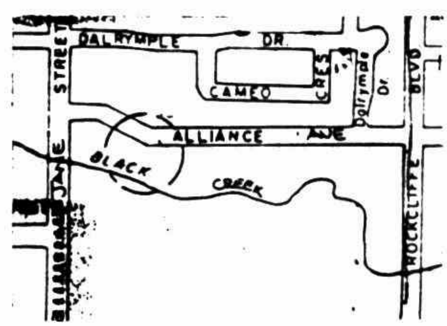


	INVERT (m.)	GRADE (%)	SIZE (mm.)	LENGTH (m.)	METHOD: 1	OUTFALL No.
STORM	99.40	9.66	300	—		L-75
SANITARY	93.02	0.28	1650	—	SCALE: 1:500	
INTERCEPTOR	—	—	100	48.64		

	INVERT (m.)	GRADE (%)	SIZE (mm.)	LENGTH (m.)	METHOD: 3	OUTFALL No.
STORM	103.97	9.66	300	—		L-75
SANITARY	104.51	0.50	225	—	SCALE: 1:500	
INTERCEPTOR	—	—	100	15.2		

FIGURE No: A-37



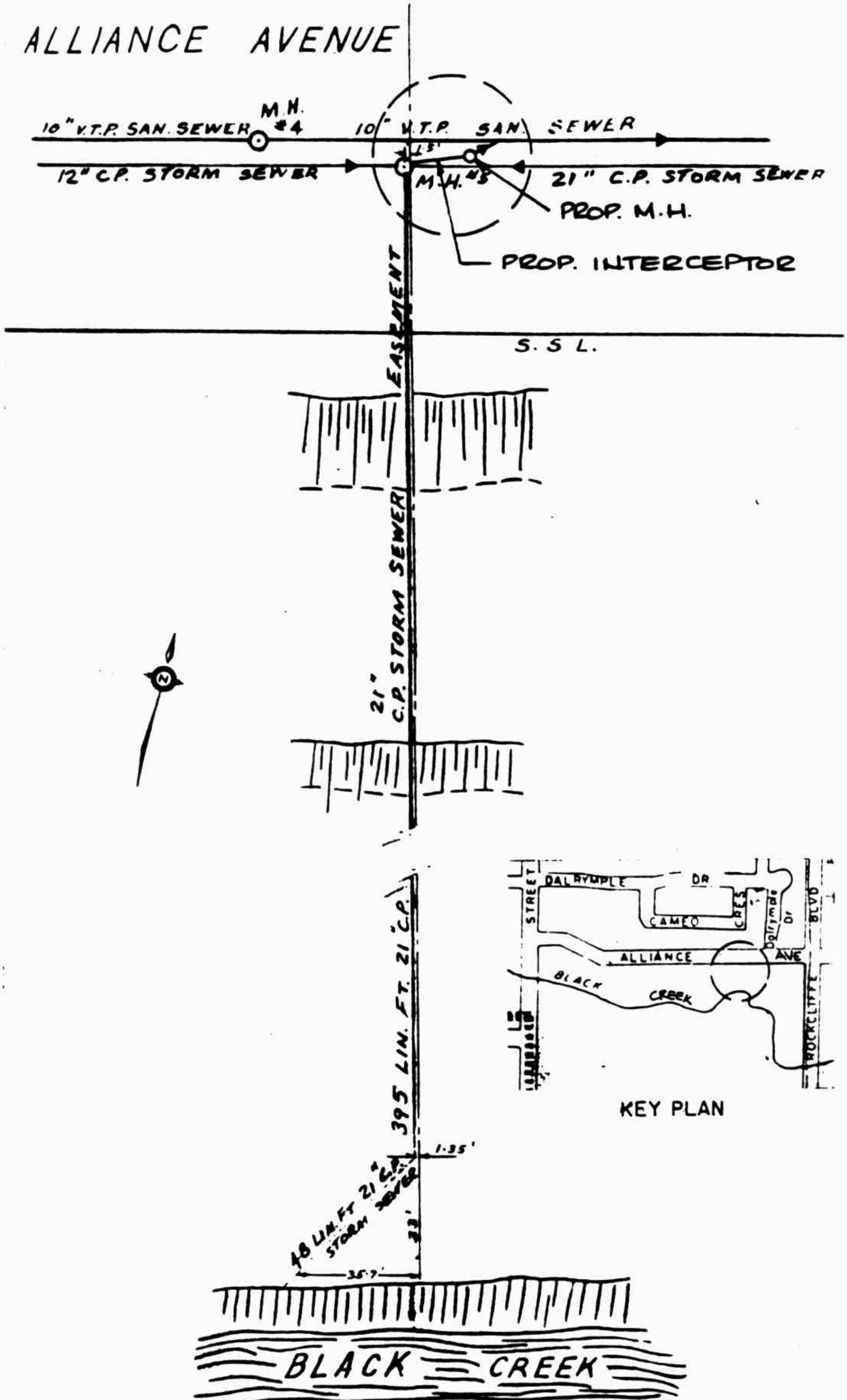


KEY PLAN

	INVERT (m.)	GRADE (%)	SIZE (mm.)	LENGTH (m.)	METHOD: 1	OUTFALL No.
STORM	101.27	2.1	750	—		L-85
SANITARY	94.71	0.28	1650	—	SCALE: 1:500	
INTERCEPTOR	—	—	100	53.34		

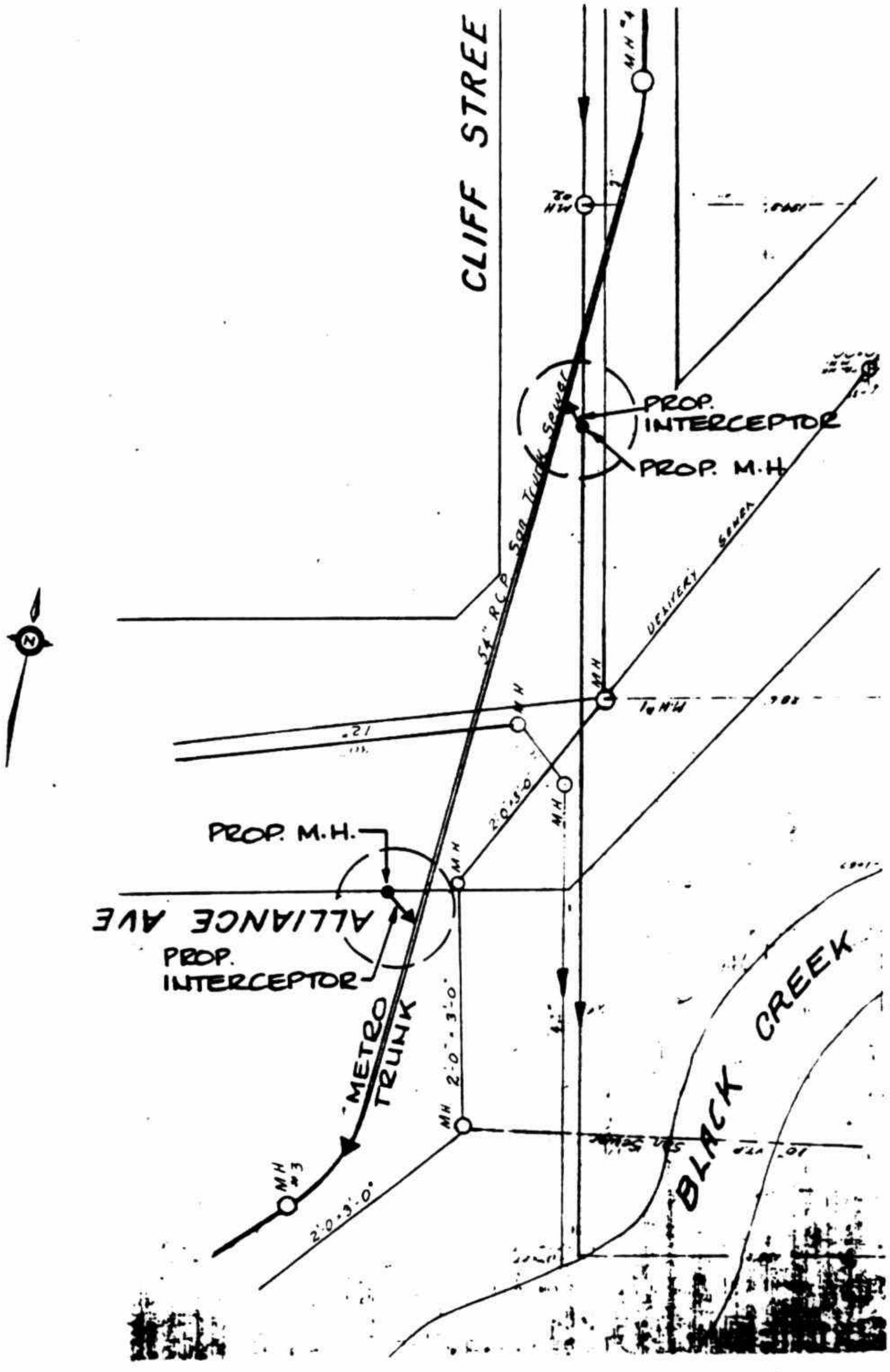
	INVERT (m.)	GRADE (%)	SIZE (mm.)	LENGTH (m.)	METHOD: 3	OUTFALL No.
STORM	101.80	3.5	750	—		L-85
SANITARY	104.08	—	250	—	SCALE: 1:500	
INTERCEPTOR	—	—	100	9.1		

FIGURE No: A-38



	INVERT (m.)	GRADE (%)	SIZE (mm.)	LENGTH (m.)	METHOD: 3	OUTFALL No.
STORM	101.83	1.7	525	—		
SANITARY	103.32	—	250	—	SCALE: 1:500	L-87
INTERCEPTOR	—	—	100	12.2		

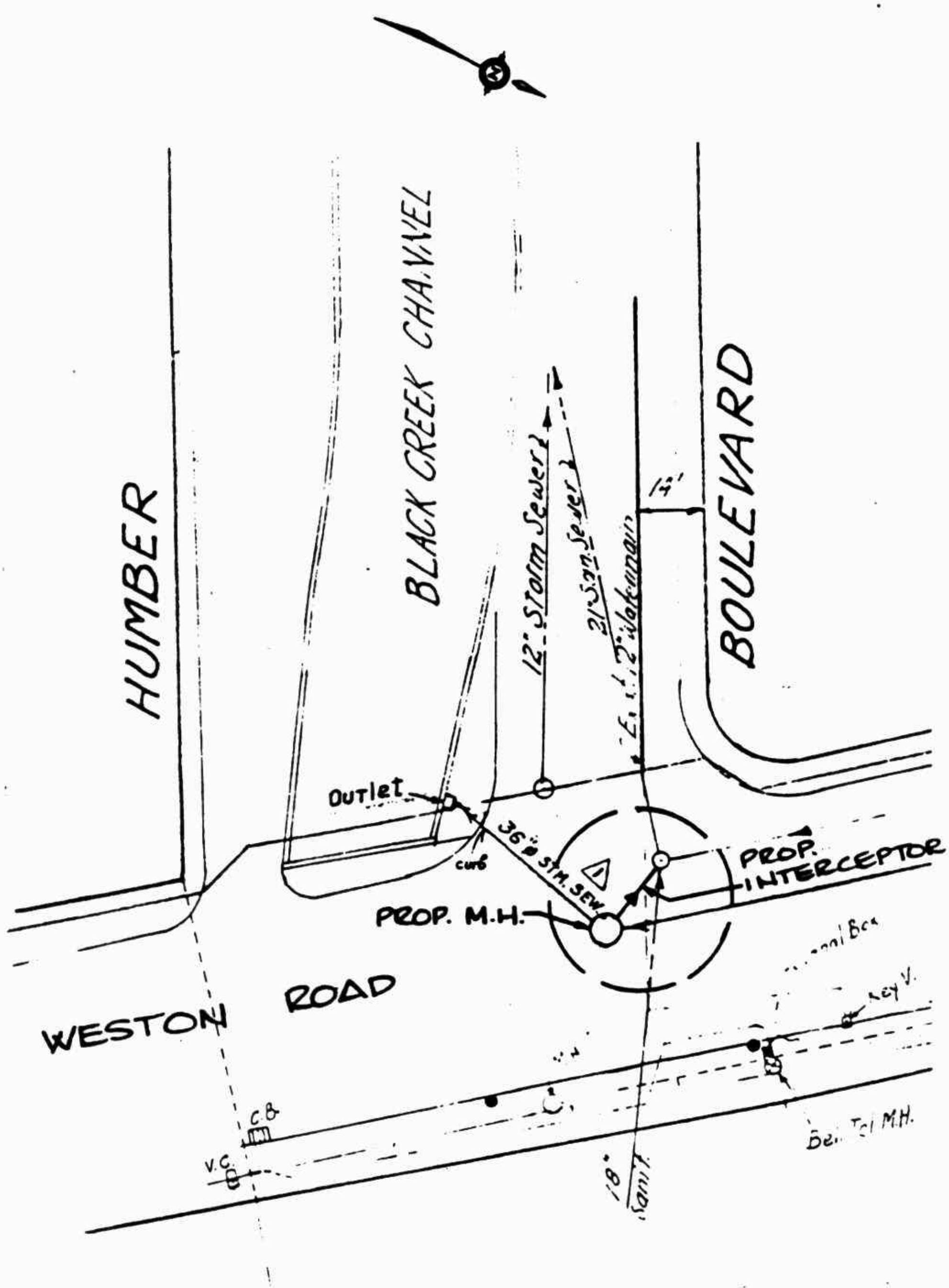
FIGURE No: A-39



	INVERT (m.)	GRADE (%)	SIZE (mm.)	LENGTH (m.)	METHOD: 2	OUTFALL No.
STORM	100.62	—	750	—		L-109
SANITARY	97.28	0.28	1350	—	SCALE: 1:750	
INTERCEPTOR	—	—	200	1.52		

	INVERT (m.)	GRADE (%)	SIZE (mm.)	LENGTH (m.)	METHOD: 2	OUTFALL No.
STORM	100.47	—	900	—		L-109
SANITARY	97.43	0.28	1350	—	SCALE: 1:750	
INTERCEPTOR	—	—	200	1.52		

FIGURE No: A-40



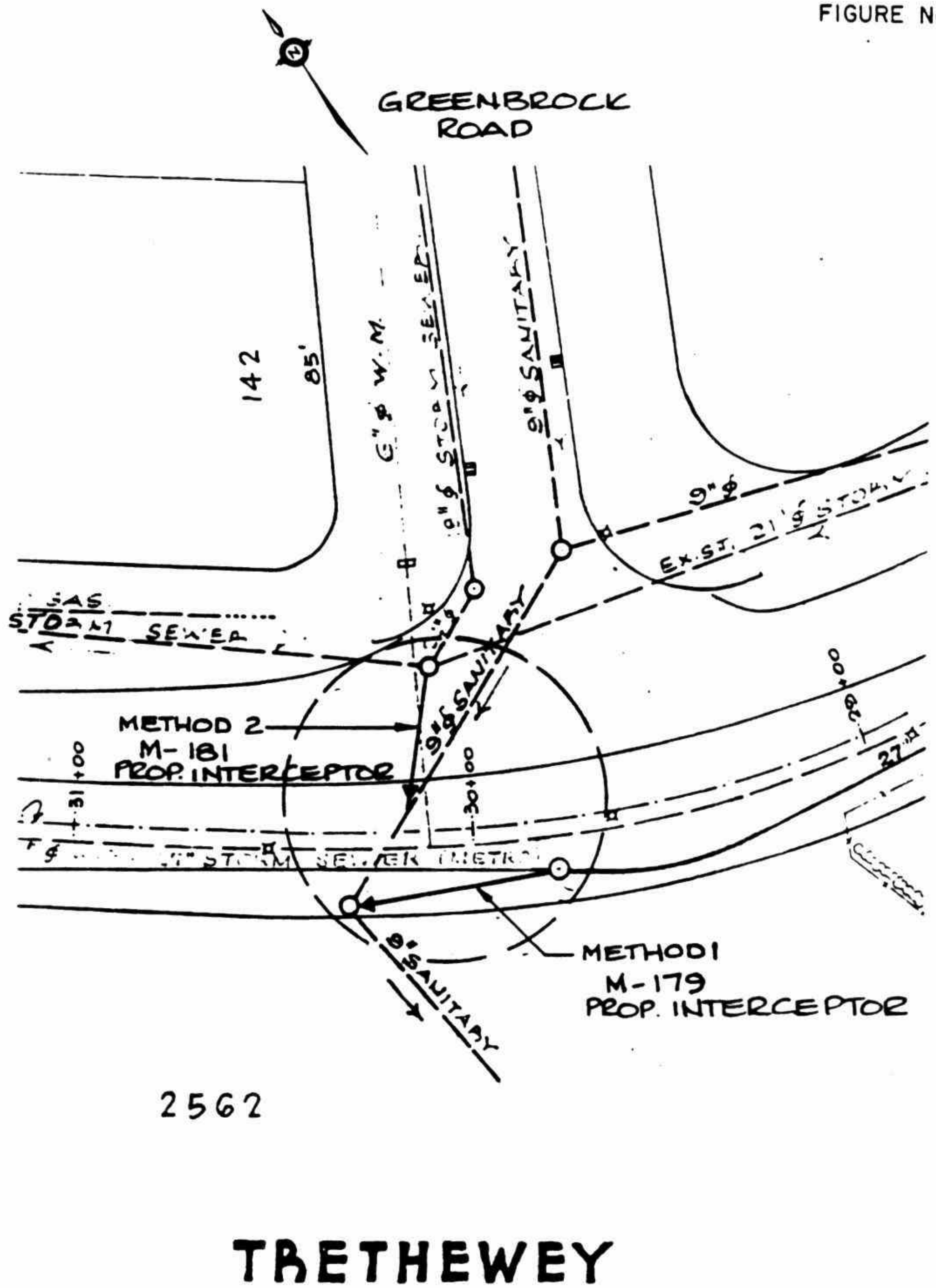
	INVERT (m.)	GRADE (%)	SIZE (mm.)	LENGTH (m.)
STORM	102.76	1.4	900	—
SANITARY	101.99	—	450	—
INTERCEPTOR	—	—	200	6.0

METHOD: 2

SCALE: 1:500

OUTFALL No. L-139

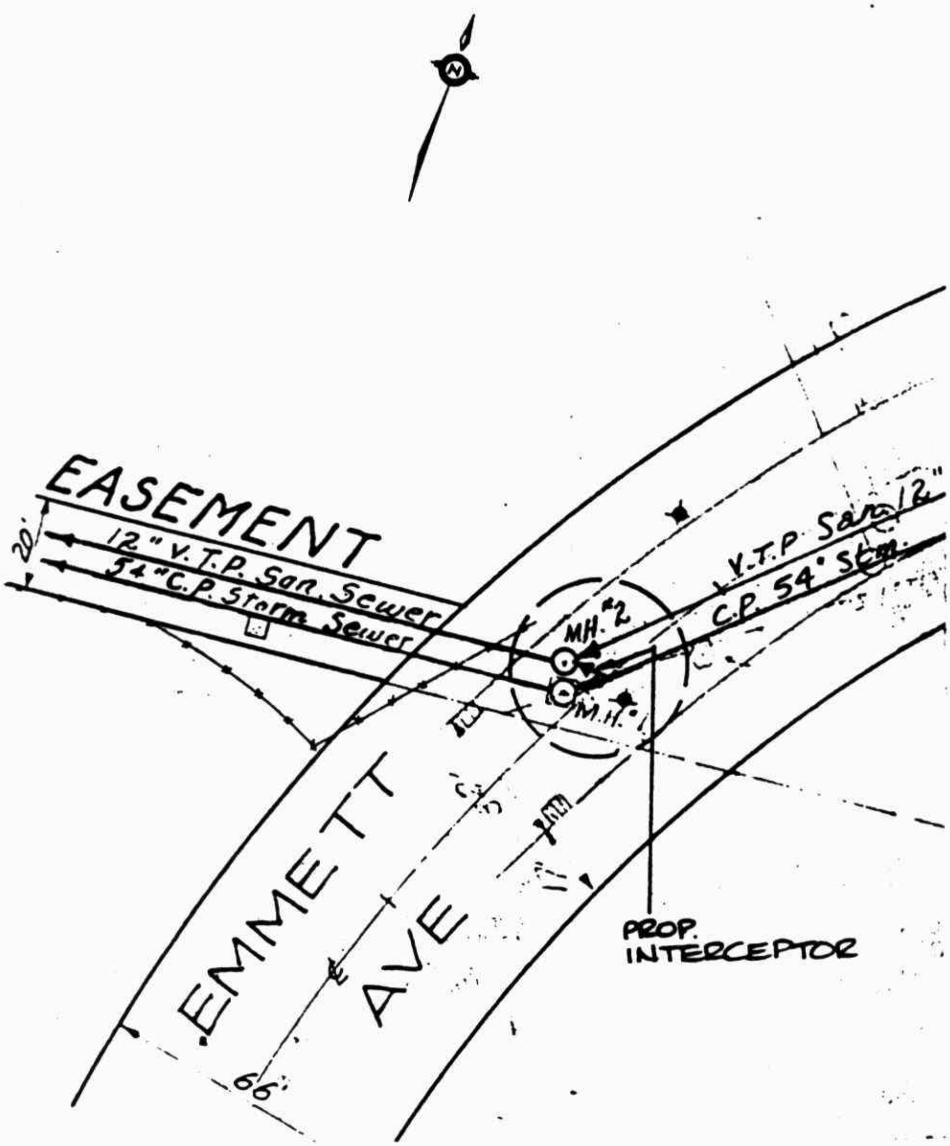
FIGURE No: A-41

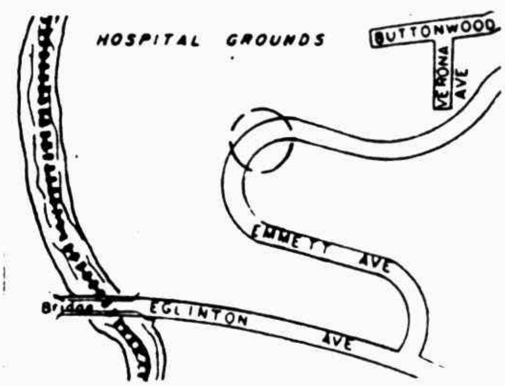


	INVERT (m.)	GRADE (%)	SIZE (mm.)	LENGTH (m.)	METHOD: 1	OUTFALL No.
STORM	109.84	—	675	—		M-179
SANITARY	106.51	—	230	—	SCALE: 1:500	
INTERCEPTOR	—	—	100	15.2		

	INVERT (m.)	GRADE (%)	SIZE (mm.)	LENGTH (m.)	METHOD: 2	OUTFALL No.
STORM	108.57	—	600	—		M-181
SANITARY	106.51	—	230	—	SCALE: 1:500	
INTERCEPTOR	—	—	100	18.24		

FIGURE No: A-42

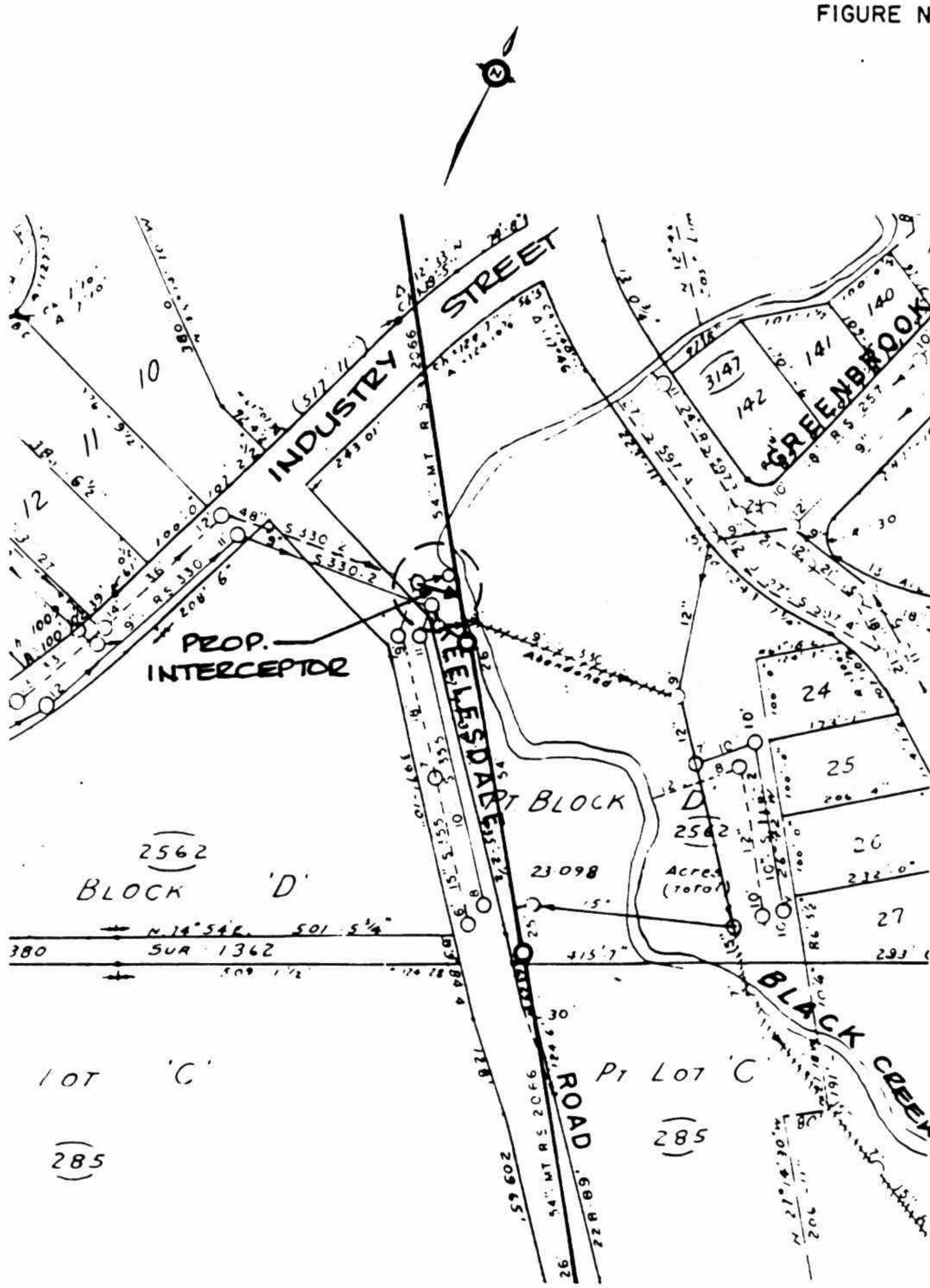




KEY PLAN

	INVERT (m.)	GRADE (%)	SIZE (mm.)	LENGTH (m.)	METHOD: 2	OUTFALL No.
STORM	104.64	0.545	1350	—		D-29
SANITARY	103.84	2.33	300	—	SCALE: 1:500	
INTERCEPTOR	—	—	100	1.0		

FIGURE No: A-43



	INVERT (m.)	GRADE (%)	SIZE (mm.)	LENGTH (m.)	METHOD: 2	OUTFALL No.
STORM	107.16	0.85	1200	—		M-175
SANITARY	—	—	1350	—	SCALE: 1:2500	
INTERCEPTOR	—	—	150	15.2		